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DISCUSSION

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

OPERATIVE ENERGY CONCEPT IN MARINE FENDERING

By Shu-t'ien Li,[1] F. ASCE

SYNOPSIS

This paper develops an operative energy concept in marine fendering; treats of variable design factors, evaluates ship impact energy, proposes design-energy criteria; determines energy-absorbing capacity of fender piles; examines mechanical properties of materials, especially greenheart, relevant to fender design; simplifies energy-design formulae; compares energy-absorbing capacities of different materials; and arrives at working formulas of easy application.

INTRODUCTION

The old-type timber-piling wharf has compatibility of strength and resilience by itself even without fendering. In modern structures of steel and concrete, the requirement for resilience and energy absorption from ship impact has to be met by additional more effective fendering.

In wharves exclusively for medium-size and smaller ships, the platform combines the functions for loading and for breasting with a length approximately equal to that of the parallel wall-sided portion of the ships. Such wharves are to be designed for the required strength as well as for absorbing the impact energy by the fendering system along their faces.

Where wharves are to accommodate exclusively for larger ships, it becomes more economical to have the breasting loads delivered to special structures or dolphins, and to set back the loading platform a few feet from

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[1] Formerly Chmn. and Chf. Engr. of the Great Northern Port Development Bd. and Cons. Engr. to Hulutao, Amoy and Kaohsiung Harbors; Cons. Civil Engr., Mobile, Ala.

the face of the wharf, so that the ship will not touch it but bear against adjacent breasting, fendered structures.

Whatever be the layout and functional requirement of a wharf or jetty, fendering is an essential component of the construction to avoid mechanical damages to both marine structure and ships due to destructive impact energy. Various types of ingenious fender systems have been proposed and used as will be described later. But after all, for general-purpose wharves and jetties, the conventional-type timber-pile fendering offers, as ever, simplicity, strength, and resilience.

In this paper will be treated potential mechanical marine damages and energy aspects in marine fendering, determination of variable design factors, evaluation of impact energy delivered by a berthing or berthed ship, how impact energy can be absorbed by fender piles in the form of internal strain energy in flexure and shear, simplified energy design formulae, comparative merits of different materials in energy-absorbing capacity, and the high capability in this respect available in the God-created greenheart (17)[2] according to mechanical test results of this timber.

MECHANICAL MARINE DAMAGES

While approaching or leaving a waterfront facility, or during the process of mooring, or during berthing periods, a ship and the waterfront structure may be subjected to various modes of heavy or light impact shock and different degrees of abrasive action. Unless suitable means are provided for deflecting or absorbing the shock and abrasion, either the structure or the ship, or both, may be severely damaged. Such damages have resulted in heavy repairs to, or renewal of, the main structure, and in denting, or the necessity of replating, the ship.

Mechanical marine damages (18) may be caused by the impact energy of berthing vessels approaching at a velocity, by their warping action during berthing, or by natural forces, while berthed, due to atmospheric and harbor basin disturbances. Among the latter, the range action deserves special notice. B. W. Wilson (8), F. ASCE (1950) states:

> "The effect of severe range action on shipping berthed alongside solid quays or jetties is always impressive and sometimes alarming. For no apparent reason a ship will describe simultaneous translatory motions in three dimensions within the compass of her mooring ropes. There have been cases, especially with large ships, where this action has been severe and prolonged enough to break all the mooring ropes and to splinter the timber fendering between the ship and the quay. On such occasions the ship's shell plating and bulkheads undoubtedly will also suffer damage of a serious kind."

He further states:

> "Most of the severe damage that occurs during range action results from the unsatisfactory cushioning medium to absorb the shock when the ship is impelled toward the quay during the on-movement. . . ."

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2 Numerals in parentheses refer to corresponding references in the Appendix.

He concludes:

". . . A ship lying at the loop end of a transverse seiche is subjected to a comparatively small lateral displacement which cannot usually exceed about 1 ft, either way from rest position, under the worst conditions, and cannot call into effective play the spring action of the ropes. Nevertheless, this amount of movement is sufficient to occasion very large impact forces between the ship and the quay. The most dangerous periodicities in this action are (as in longitudinal range action) in the range from about 1 min down to 20 sec (for most ships of commercial sizes, say, from 5,000 tons displacement to 30,000 tons displacement). Of greatest importance is ship size, since the force of impact increases virtually as the square of its mass. Berths located along the longer side of a dock will probably always be more susceptible to the troubles inherent in transverse ranging than berths along the shorter side.

"Transverse impact is undoubtedly the most serious features of range action. Initially tight ropes will not be of much assistance in this case, ... If the evil is to be endured, the possible protective measures are the use of shock-absorbing fenders or of bow and stern anchors to hold ships off the quays."

FENDER CLASSIFICATION, ADAPTABILITY, AND ENERGY ABSORPTION

A fender system is an important, indispensable, though supplementary, component of water-front facilities, designed to absorb shock from impact and to resist glancing or grinding abrasion for the purpose of protecting both the structure and floating craft from mechanical damages. It may consist of fender piles driven in front of a structure; or timbers in vertical or nearly vertical planes fastened to the face of a wharf, sea wall, or other water-front structure, with chocks and spacing timber between them; or any other type that will be described hereunder.

As size of ships increases in recent years, the design of heavy-duty fenders has become ever more important. For convenience of increasing capacity and resilience, and of replacement, fendering may best be designed in units or panels. Their component parts are in general composed of (a) the rubbing face, (b) the structural frame and supports, and (c) the resilient units.

The rubbing face is usually made of hardwoods such as white oak and other exotic species among which Demerara (17) greenheart is the best. Vertical structural frames may be of steel or timber piles. Instead, certain types of fenders may be directly attached to the face of the wharf or hung from its deck. Neither structural steel nor reinforced concrete is as efficient as timber in cantilever spring action when used as vertical piles fixed at the bottom, or in springing flexural action when both top and bottom are supported. Unless special resilient-type units are used, the fenders themselves must have resilient quality.

Different types of fenders have been used to adapt them to diverse purposes and types of water-front structures, to various sizes and types of ships, and to different impact shocks and abrasive action. The functional adaptability of each principal type of fenders, and the manner in which the work or energy of the impact of a moving vessel on a water-front structure is absorbed may

be described together with a broad fender classification according to kind of material, way of installation, or device for absorbing energy.

(1) Conventional-type timber-pile fenders are usually used for general-purpose wharves, piers, and jetties. They form the vertical frame and have high resilience and spring action. Impact energy is absorbed directly by flexural and shearing strain capacity of fender piles and to some extent indirectly by the structural members behind them. They are driven either singly or in clusters (dolphins), and may be either with or without a cap and upper and lower wales, as location or circumstances may require. When horizontal loads are carried to the lower parts of the piles, their embedment in the soil must be sufficient to transfer the horizontal forces to the soil. They represent the simplest type and have more inherent advantages than any other type. Their batter outward, if used, should be kept at a minimum and in all cases far less than 1 in 6 beyond which they themselves will likely suffer breaking damage.

(2) Floating-type fenders are, in general, provided in water-front facilities for berthing large vessels or super-tankers, or where there is large variation of water levels. In the larger versions they may be fendered pontoons; in medium size, floats with stiffeners inside and wrapped with ropes or rubber tubes outside; and in smaller, crude versions, simply wood rafts or round logs. Originally, they were introduced to keep the ship away from the wharf's face. They also serve as an additional cushion aiding the fenders in absorbing the ship's impact. For this purpose, they may be installed as an aftermath in an otherwise inadequately fendered wharf when it is difficult to drive extra fender piles later as more energy-absorbing capacity becomes necessary. The impact energy is absorbed by deformation of the floating fenders, or "camels" as they are called colloquially. A replaceable fendering surface, or metal rubbing strip, is provided on the vertical fender piles, if any, behind them to withstand abrasion, unless greenheart piles are used.

(3) Hung-type fenders have been mainly used for preventing abrasion to either filled cellular or open-type piers where the expected impact is insignificant. They consist of short lengths of timber or steel members fastened rigidly to the outboard sides of the main structure so that they can be easily replaced when excessively damaged or worn. They are, however, not effective in absorbing heavy impact energy because of their limited lateral deflection and hence low capacity of internal strain energy.

(4) Resilient-type fenders have been used to absorb the impact energy due to large ships, and to protect heavy and rigid water-front structures, where there is not much fluctuation of water level. They form a deflecting, cushioning system that absorb the kinetic energy of a moving vessel, and consist of buffers or springs placed between the outboard fendering surface and the main structure. The resilient media that absorb impact shock by compression include:

(a) Special hardened steel springs of single, double, or multiple coils depending on the capacity (up to 50 tons or more) required;

(b) Laminated steel springs;

(c) Rubber buffer blocks vulcanized to steel mountings, functioning like springs;

(d) Rubber buffer spirals;

(e) Hanging rubber-tube segments consisting of hollow cylinders of rubber on steel cable supports, either horizontal or vertical;

(f) Old motor-vehicle tires hanging over the side; and

(g) Hydraulic-type shock absorbers.

If parallel or nearly parallel ship approach is stipulated, intermittent panels of spring fendering will suffice. The spaces between them may be lined with timbers for occasional contact of smaller ships or barges. Steel springs are usually contained in a steel housing and protected by non-corrosive metallic coating of nickel or cadmium and by periodic greasing. They are preferably located above the reach of water. Rubber blocks in front of breasting concrete cap are easy to make provisions for additions. The serviceable life of rubber in salt water may be extended by neoprene coating. As a safe precaution it is always advisable to avoid cut down the static resistance of a pier and arrange for long-travel springs or sophisticated devices. The latter might be missed entirely by a ship on impact.

(5) Suspended-type fenders have been used where the installation of long fender piles is too expensive, or where there is large seasonal variation in the water level of a river. A suspended fender has the advantage of being easily replaced in an emergency. Inconvenience may, however, occur where the water front is frequented by a large variation of size of berthing vessels. They depend on the heavy gravity weight of the suspended blocks, usually of concrete, to absorb the major portion of the kinetic energy of a moving vessel. Such blocks are suspended from the water-front facility by means of steel links or chains. The impact pressure resulting from a berthing vessel not only moves the weight inward but also lifts it upward, thereby expending a large portion of the kinetic energy and reducing the net lateral force transmitted to the main structure.

(6) Retractable marine fender system (15) has been used to achieve large energy absorbtion, economy, and permanence. It consists of a continuous timber vertical member and horizontal wale frame suspended by bolts in slotted brackets fastened to the pier.

The previous brief description of various types of fenders suffices to show where timber-pile fenders stand in the domain of fendering. While each type has some special advantages, the timber-pile fenders, especially of greenheart, have proved to best meet the essential requirements, specified in a later section, for the fendering of general-purpose wharves and piers.

MEDIA OF ENERGY ABSORPTION GENERALLY NOT CONSIDERED

In the normal procedure for designing fenders, several available media of energy absorption such as (1) elastic deformation of the vessel, (2) yawing and rolling of the vessel, (3) deformation and movement of the wharf structure, (4) displacement of water between vessel and quay, (5) plastic deformation of the ground caused by deflection of piling, (6) wave produced by moving ship, and (7) heat generated by the impact, are generally not considered, not only for conservative reasons but also for the expedient reason that they are not readily

evaluable. It is advisable to regard them as counterbalancing inevaluable forces and disturbances.

MODES OF ENERGY ABSORPTION TO BE AVOIDED

There are certain modes of energy absorption that both port engineers and ships' captains will attempt to eliminate or reduce to a minimum. These include: (1) plastic deformation of ship hull, (2) denting beltings of belted vessels, (3) biting of ship into a water-front structure, and (4) cutting by bow into the structure. They are, however, not always entirely avoidable, and the energy so expended has contributed in many a case to increasing the factor of safety of the main structure which has been thereby saved from destruction.

ESSENTIAL REQUIREMENTS FOR THE CONVENTIONAL-TYPE TIMBER-PILE FENDERING

Essential and desirable requirements for the conventional type of fendering for general-purpose wharves, quays, piers, and jetties are enumerated as follows:

(1) Adaptability to both wall-sided and belted vessels to berth alongside;

(2) Long serviceable life, low maintenance, and least renewal;

(3) Minimum capitalized or annual cost;

(4) High absorbing capacity for impact energy so as to eliminate damages to the main structure;

(5) Appreciable elastic movement so as to eliminate damages to the berthing ship;

(6) Capability of absorbing inclined impacts and rubbing forces so as to eliminate damages to fendering;

(7) Having jointly with the main structure sufficient static resistance and mass to cause plastic deformation of ship hull in order to save the main structure if hit by an abnormal impact;

(8) Capability of absorbing work from a bumping vessel at exposed berths;

(9) Avoidance of over rigidity and stiffness; and hence

(10) Relief of ship captains' psychological fear of bumping against over-rigid fenders that has sometimes led to the choice of casting off.

All these requirements can be better met by timber, and best met by greenheart, pile fendering.

VARIABLE DESIGN FACTORS

These factors include tidal fluctuations, atmospheric disturbances, harbor-basin disturbances, exposed areas and displacement of berthing vessels, approaching velocities of vessels, and warping of vessels into their berths.

(1) Tidal fluctuations affect the vertical movement of berthed vessels alongside a fender system. When the tidal range becomes large, to better resist lateral loads, a lower wale-and-chocking system is usually provided.

(2) Exposed areas of berthing vessels refer in particular to their surface areas above and below the ship's waterline respectively normal to the direction of winds and currents.

(3) Atmospheric disturbances refer to those above the ship's waterline. They manifest from ordinary winds to those most violent associated with hurricanes, cyclones, and typhoons. Wind loads are normally determined from Weather Bureau records of the highest wind velocity for a period of 5 min.

Where past records of wind velocities are available at nearby weather stations, the wind load in lb per sq ft on the vertical projection of the exposed area (A_w) normal to the direction of the wind may be obtained by the writer's linear equations formulated by equating velocity and pressure heads for an air density of 0.0765 lb per cu ft at 59°F, 76 cm mercury column, using the upper limit of the probable form factor of 1 3/4 to 2 for the long and almost prismatic silhouette of the ship above waterline. If V = wind velocity in mph, p_w = the wind load intensity in psf, and P_w the total wind load in lb, then: for V = 60 to 80 mph,

$$p_w = \frac{7}{10} V - 23 \qquad (1a)$$

for V = 80 to 100 mph,

$$p_w = \frac{9}{10} V - 39 \qquad (1b)$$

for V = 100 to 120 mph,

$$p_w = \frac{11}{10} V - 59 \qquad (1c)$$

and

$$P_w = p_w A_w \qquad (1d)$$

In inner harbors, these wind-load intensities may be reduced by a schedule according to the height of observation above the water surface: no reduction if 50 ft or lower, 5% over 50 ft to 100 ft, 10% over 100 ft to 150 ft, 15% over 150 ft to 200 ft, and 20% over 200 ft.

If berths are exposed to the open sea with a long fetch over water, no reduction of wind velocity should be made, as wind on water generates a progressive wave which attacks the ship together with the wind. It is, therefore, a simplified, safe practice to use the full observed wind velocity and neglect the effect of the progressive wave.

For extremely high wind velocities in the order of over 100 to 120 mph that may occur during short periods of the severest hurricanes, instead of designing the fenders to sustain such rare wind loads, it is advisable to require ships anchor off to avoid being damaged, or fill their ballast compartments to reduce freeboard and wind exposure.

(4) Harbor basin disturbances refer to those below the ship's waterline such as waves, dynamic pressure of currents, drag force or frictional resistance, floating ice, ground swells, and those caused by passing ships for berths adjacent to navigation channels. Though both natural and artificial harbors are generally located in sheltered or protected waters, waves have to be considered except at inner harbor berthing facilities completely free from seiches and waves. Large floating ice is not expected to exist even in cold climates if the harbor is kept open by icebreakers. Ground swells and disturbances caused by passing ships are not readily evaluable, and hence they may be considered as counterbalanced by "media of energy absorption generally not considered." The remaining forces that can be evaluated with a reasonable degree of accuracy are the dynamic pressure of currents, the

drag force of frictional resistance though it may be comparatively insignificant, and the transverse impact under the stimulus of a seiche.

(a) Dynamic pressure of currents is generally appraised as equivalent to twice the velocity head to allow for the dynamic effect. Hence, if v_c = velocity of the currents in knots, (1 knot = 1.69 fps); g = gravity acceleration = 32.2 fps per sec; h = head in ft; p_c = intensity of pressure in psf; w = unit weight of sea-water, taking as 64.4 pcf; C = a coefficient for bilge shape = 1 for longitudinal hull to 3/4 for a rounded bilge; A_c = projected area in sq ft normal to the direction of currents; and P_c = total dynamic pressure in lb; then statically,

$$\frac{p_c}{w} = h = C\left(\frac{V_c^2}{2\,g}\right)(1.69)^2 \quad \text{(2a)}$$

whence

$$p_c = 2.86\ C\ V_c^2 \quad \text{(2b)}$$

and dynamically

$$p_c = 5.7\ C\ V_c^2 \quad \text{(2c)}$$

Hence,

$$P_c = p_c\ A_c = 5.7\ C\ V_c^2\ A_c \quad \text{(2d)}$$

(b) Drag force or frictional resistance of the submerged hull surface area may be evaluated by Froude's equation

$$R = f\ S\ V_c^2 \quad \text{(3)}$$

in which R is the resistance in lb, S denotes the submerged hull surface area in sq ft, v_c represents the current velocity in knots, and f is a coefficient depending on length of vessel ≅ 1/100 under ordinary average conditions.

(c) Transverse impact under the stimulus of a seiche will have the worst condition, according to Wilson (8), when the ship has a clearance between itself and the fenders exactly equal to the amplitude of the on-movement, for then the impact will occur just as the accelerations of the ship and the water mass reach their peak. This impact is represented by the first term of the following equations after Wilson under reasonably simplified set of conditions. The second term accounts for the additional force of the inward pull of the ship's bow or stern ropes, if the ship also completes a lunge fore or aft at the instant of impact. Combining the two terms, the maximum impact force transverse to a quay from a ship lying along the longer side, D, of a dock is given by

$$P_{(max.)} = \pi\ Y_o\ W\left(\frac{2\,\pi\ M_y^2\ A_y}{B^2} + \frac{M_x\ A_x}{X\ D}\right) \quad \text{(4)}$$

and the maximum impact force transverse to a quay from a ship lying along the shorter side, B, of a dock is given by

$$P_{(max.)} = \pi\ Y_o\ W\left(\frac{2\,\pi\ M_x^2\ A_x}{D^2} + \frac{M_y\ A_y}{X\ B}\right) \quad \text{(5)}$$

in which x-axis is horizontal along the long side of the dock; y-axis is horizontal at right angles to the long side of the dock; Y_o is the perpendicular distance in ft between longitudinal centerline of ship at rest and face of quay (face of fenders); W represents the displacement tonnage of the ship; M_x, M_y

is an integer defining the nodality of the seiche respectively in the longitudinal and transverse direction of the dock; A_x, A_y refers to the maximum vertical amplitude (half range) in ft respectively of the longitudinal and transverse seiches; D is the length, or the longer side, in ft of the dock basin; B represents the width, or the shorter side, in ft of the dock basin; X denotes the maximum projection in ft of the bow mooring line along the quay at which the ship is lying; and $P_{(max.)}$ is the transverse impact force in tons.

(5) Displacement and effective mass of berthing vessels refer respectively to the actual displacement weight and the effective mass associated with the impact energy.

Just how much of the gross kinetic energy of a berthing ship at a given approaching velocity is delivered to the fender system at any point of impact depends on how much of the entire mass is effectively acting. When berthing a larger wall-sided vessel, half (9) of its mass or displacement is generally considered active. This is because of a considerable portion of its length is built off the same template, the resultant vessel can contact the fender system over a long length or near either end. If an end impact takes place, the mass center is free to continue moving, and hence only a fraction of the whole mass is acting. On the other hand, if she comes in parallel to the fendered face, there will be comparatively little load acting per ft run. Thus, if the fender can resist an end coming in first, it will surely be able to resist a distributed impact over a considerable length.

In the case of an end impact taking place with non-parallel docking approach, the velocity vector (v) normal to the fender face does not coincide with the reaction, and consequently only a part of the kinetic energy manifests as impact energy while the rest rotates the ship to deliver a secondary contact blow to the fenders. Although velocities with a non-parallel approach may generally be greater than with parallel approach since in the latter case the ship is usually under better control, the effective mass (M_e) will be much reduced. If a vertical axis is considered as passing through the point of contact (0), r is the distance in ft from this point to the mass center of the ship, θ is the horizontal angle in deg the line r makes with the front face of fenders, ρ is the mass radius of gyration in ft of the ship about 0, and ω is the angular velocity in radians per sec of rotation of the ship about 0; then according to the "principle of conservation of momentum," the moments of momentum instantly before and after the impact contact are almost equal to each other. Hence,

$$M\,v\,(r\cos\theta) = M\,\rho\,\omega\,(\rho) \quad \ldots\ldots\ldots\ldots\ldots (6)$$

giving

$$\omega = v\,r\cos\left(\frac{\theta}{\rho^2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The effective kinetic energy will be

$$\frac{1}{2}\,M_e\,v^2 = \frac{1}{2}\,M\left(V^2 - \rho^2\omega^2\right)$$

$$= \frac{1}{2}\,Mv^2\left(1 - r^2\cos^2\,\frac{\theta}{\rho^2}\right) \quad \ldots\ldots\ldots (8)$$

whence the "effective mass" is given by

$$M_e = M\left(1 - r^2\cos^2\,\frac{\theta}{\rho^2}\right) \ldots\ldots\ldots\ldots\ldots (9)$$

In case the mass radius of gyration of the ship about her center of gravity = 1/4 of her length (L) and r = 1/3 of her length, θ = 27 5/6 degrees

$$\rho^2 = \left(\frac{1}{16} + \frac{1}{9}\right) L^2 \quad \ldots \ldots \ldots \ldots \ldots \quad (10)$$

and

$$M_e = \frac{M}{2} \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (11)$$

For all θ smaller than $27\frac{5}{8}$ degrees, M_e will be less than M/2 if the above values of ρ and r remain the same.

While it is reasonable to take half the mass as acting in the case of wall-sided vessels of 20,000-ton class or over, the effectively acting proportion of the entire mass will increase as the displacement tonnage decreases, and this proportion may increase to nearly the full mass in the case of belted vessels of the 2,000-ton class or under.

The belted vessels are generally built more curved in plan and sometimes with completely curved beltings capable of delivering the whole impact as a concentrated load on the face of the fender piles. These vessels are sturdy and can resist a much greater localized reaction than a wall-sided vessel without suffering from plastic deformation. Consequently, not only they may deliver the full gross kinetic energy to the fender system, but also usually they berth at a much higher speed, thus making their kinetic energy as high as, or even slightly higher at times than, a large wall-sided vessel of ten-times the tonnage displacement.

(6) Approaching velocities of vessels depend mainly on the methods of docking, but may be affected somewhat by wind and current. For ships of 10,000-ton displacement or over, in restricted harbor basins where the ship is usually brought in by tugboats, the normal component of velocity may be as low as 0.00+ to 0.25 ft per sec. If there is free approach to the wharf and the ship comes in under its own power, the normal component of velocity may be in the range from 0.25 to 0.50 ft per sec. Smaller ships generally come in at much higher velocities. For coastwise ships of 2,000 long tons or under, the normal component of their velocities approaching the wharf may be as high as twice the previous figures or even slightly higher.

(7) Warping of vessels into their berths produces a rather indeterminate magnitude of loads, which depend on such variables as mass and velocity of berthing vessels as well as the resiliency of piers and fenders. It may be taken as counterbalanced by "media of energy absorption generally not considered."

KINETIC-ENERGY AND OBLIQUE-LOAD EVALUATION

The kinetic energy approach is preferred for evaluating the impact energy on a fender system delivered by a ship with an approaching velocity to its berth and with an added velocity from the waves if there be any, while the oblique-load approach is advisable for use to appraise the normal and tangential forces due to wind, current, drag and impact transmitted by a berthed vessel to the fenders.

(1) The Kinetic Energy Approach.—In this method, the magnitude of the impact energy is evaluated by the basic equation of kinetic energy. Let W

= displacement of ship in long tons; v = vector component of ship's velocity normal to the face of the fenders, in fps; K. E. = kinetic energy in ft-lb. Then, with gravity acceleration (g) as defined under "dynamic pressure of currents,"

$$K.\ E. = \frac{1}{2}\ M\ v^2 = 2{,}240\ \frac{W\ v^2}{2\ g} = 34.8\ W\ v^2 \ \ldots\ldots\ldots \quad (12)$$

whose evaluation depends on the selection of the maximum weight of ship, her effective mass acting, and her approaching velocity which in turn depends on the methods of docking as stated under "approaching velocities of vessels."

In case of surge motion in progressive waves and also for the case of transverse impact of ship's approach combined with effects of a beam sea of either progressive or standing waves, according to Wilson (16), but using more simplified notations, the kinetic energy (K. E.) may be expressed by increasing the transverse velocity of the approaching ship by the added velocity from the waves as

$$K.\ E. = \frac{1}{2}\ M'\ (V' + V)^2 \ \ldots\ldots\ldots\ldots\ldots\ldots \quad (13)$$

in which V is the transverse uniform velocity of approach in ft per sec; V′ denotes the added variable sway velocity in ft per sec due to the wave; and M′ represents the virtual mass, the mass of the ship increased by the hydrodynamic effect of displaced water, which is given by

$$M' = M\left(1 + \frac{\pi\ B}{16\ D}\right) \ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

in which B is the length of the beam in ft, and D denotes the draft in ft. For an open structure and transverse approach combined with a progressive beam wave, V′ is defined by

$$V' = \frac{A\ g}{\sigma\ D}\ \frac{\sinh\ (k\ d)\ -\ \sinh\ (k\ s)}{\cosh\ (k\ d)}\ \frac{\sin\left(\frac{k\ B}{2}\right)}{\frac{k\ B}{2}}\ \sin\ \theta \ \ldots\ldots\ldots \quad (15)$$

in which σ is angular frequency = 2π/period; d denotes wave depth, in ft; s represents clearance under the keel, in ft; k is wave number = 2π/wave length; θ is phase number; and V′ becomes maximum when sin θ = 1. Eq. 15 for V′ does not apply to the case of a solid structure where the waves reflect and form a standing wave system. By applying specific values, the sway velocity, the virtual mass, and hence the kinetic energy may be determined.

(2) The Oblique-Load Approach.—In this method, the normal and tangential components of wind, current, drag, and impact forces acting on the fender face may be evaluated by the formulae given under "variable design factors." Let θ_w and θ_c be, respectively, the angle in degrees the direction of wind and current forces makes with the normal to the fender face, N = the normal component in kips, and T = the tangential component in kips, then

$$N = p_w\ A_w\ \cos\ \theta_w + \left(p_c\ A_c + R\right)\cos\ \theta_c \ \ldots\ldots\ldots\ldots \quad (16a)$$

and

$$T = p_w\ A_w\ \sin\ \theta_w + \left(p_c\ A_c + R\right)\sin\ \theta_c \ \ldots\ldots\ldots\ldots \quad (16b)$$

in which N is the normal or transverse component that will produce flexural and shearing strain in the fender system, and T the tangential component that will cause rubbing action on the face of fenders.

As an example of computing normal and tangential forces acting on the face of a timber-pile fender system, take a freight ship of the 20,000-gross-ton class whose overall length is 700 ft with her parallel wall-sided length 610 ft, beam 100 ft, draft 35 ft, loaded to 86% of her displacement capacity, approaching her berth at 1/2 fps; and during the berthed period, there is no appreciable seiche despite wind at a velocity of 100 mph at a height of over 100 ft above the water surface, and current at 2 knots, while the area above waterline normal to the wind is equal to 14,000 sq ft, that below the waterline normal to the current 18,000 sq ft, and $\theta_w = 0°$, $\theta_c = 30°$. From Eqs. 1 to 3, the wind load and dynamic pressure of currents are

$$p_w = 0.9\left[\frac{9}{10}\ (100) - 39\right](14) = 643 \text{ kips}$$

$$p_c = 5.7\ (0.88)\ (2)^2\ (18)\ (0.866) = 361\ (0.866) = 313 \text{ kips}$$

and the frictional resistance, being comparatively small in this case, may be neglected. Thus, by Eqs. 11,

$$N = 643 + 313\ (0.866) = 914 \text{ kips}$$

$$T = 0 + 255\left(\frac{1}{2}\right) = 128 \text{ kips}$$

and the lateral load per lin ft of berth = 914/610 = 1.5 kips.

If $\theta_w = \theta_c = 60°$, $A_w = 14{,}000 \cos 60°$, $A_c = 18{,}000 \cos 60°$,

$$P_w = 643 \cos 60° = 322 \text{ kips}$$

$$p_c = 361 \cos 60° = 181 \text{ kips}$$

whence

$$T = (322 + 181) \sin 60° = 436 \text{ kips}$$

Hence, the tangential or rubbing load may be as high as nearly one-half of the lateral or normal load.

The kinetic energy of the said berthing ship, assuming no surge motion, is given by Eq. 12 as

$$K.\ E. = 34.8\left(\frac{1}{2}\right)(20{,}000)\ (0.86)\left(\frac{1}{2}\right)^2 = 74.8 \text{ ft-kips}$$

A 2,150-ton vessel berthing with an approaching velocity of 1 fps may deliver the same kinetic energy at a contact point, for in this case

$$K.\ E. = 34.8\ (2{,}150)\ (1)^2 = 74.8 \text{ ft-kips}$$

If there were a normal contact at resilient fender buffers which are compressed by d in. and the impact load increases from zero to a maximum $(N_{max.})$, the average impact load $(N_{av.})$ would be given by

$$N_{av.}\ \frac{d}{12} = K.\ E. \quad \dots\dots\dots\dots\dots\dots \quad (17)$$

whence $N_{av.} = 12(74.8)/d = 898/d$ kips, and $N_{max.} = 2\ N_{av.} = 1{,}796/d$ kips. When $d = 11\frac{1}{4}$ in., $N_{max.} = 160$ kips. If there are 8 fender buffers in 140 ft, the maximum normal load at each buffer will be 20 kips, requiring 10-ton buffers.

Eq. 17 can only be valid if there is direct compression. When the impact energy is absorbed by internal strain energy due to bending and shear, the

method given later under "Basic Relations of Energy-Absorbing Capacities" will apply.

DESIGN LOAD AND ENERGY CRITERIA

Either the oblique-load or the kinetic-energy approach may be further facilitated by referring to established design-load practices and by formulating design energy criteria; the latter is preferred when fender piles are proportioned directly by their strain-energy capacity.

(1) Lateral or Normal Load.—In the previous example, the evaluation of the lateral load on the face of fenders for a freight ship of the 20,000-gross-ton class, when the wind velocity is 100 mph and current velocity 2 knots, yields a result of $1\frac{1}{2}$ kips per lin ft of berth. This figure may be accepted as the usual load for the design of conventional-type timber-pile fenders for general-purpose wharves, quays, piers, and jetties for berthing general cargo ships. But at the outer ends and at exposed corners of water-front facilities, larger concentrated loads may occur as a result of approaching impact and warping loads of vessels moving into their berths.

For practical design purposes in the U. S. Navy, the Bureau of Yards and Docks has found that satisfactory results are achieved by using a uniform lateral load against the fender system for various classes of ships, as follows:

Class of ship	Lateral load, per lin ft of berth (lb)
Submarines, destroyers, and other small craft	1,000
Cruisers and Cargo ships	1,500
Battleships and escort carriers	2,000
Large aircraft carriers	2,500

The Bureau considers that these values are adequate at locations where the maximum wind velocity is over 60 mph and where the currents are in excess of 2 knots. When the maximum wind velocity is 60 mph or under, the Bureau suggests that a reduction of 20% may be made in the values of lateral load given previously.

It may be noted that the lateral load computed in the previous example for a freight ship agrees well with the Bureau's established value for cargo ships, when there is no appreciable seiche.

In cases where no pertinent data are available for computing the lateral loads, and where an equivalent class of merchantman for which the water-front facility is to serve, can be ascertained corresponding to a class of ship designated by the Bureau, its established values may be accordingly used as the minimum for sake of expediency.

(2) Tangential or Rubbing Load.—In the previous example of evaluation, the tangential or rubbing load is found to be nearly one-half of the maximum lateral or normal load. In cases where no pertinent data are available for computing the tangential or rubbing loads, they may then be taken at least as one-half of the Bureau's established values for lateral loads.

(3) Kinetic Energy.—In the previous example, the evaluation of kinetic energy that may be delivered on a fender system by a freight ship of the 20,000-gross-ton class, loaded to 86% of her displacement with half of her

mass acting and approaching at a velocity of 1/2 fps, yields a result of 74.8 ft-kips. The same energy would result if the ship were fully loaded with half of her mass acting and approaching at a lower velocity of about 0.46 fps.

It has also been shown that the same energy will result for a 2,150-ton class ship with all of her mass acting, approaching at a higher velocity of 1 fps.

As the impact energy is proportional to the square of the approaching velocity, the energy level will increase considerably if the velocity is only slightly increased. Using the data for the 20,000-ton class ship of the previous example, the energy level would be tripled if the approaching velocity were only increased to 0.866 fps. As the approaching velocity should be under control and it is uneconomical to design a water-front facility and its fender system for uncontrolled accidental energy levels, it will be reasonably conservative to design the conventional-type timber-pile fenders for berthing freight ships from 2,000-ton class to 20,000-ton class to resist a normal impact energy level of 75 ft-kips at allowable working stresses for transient load, and to be able to withstand the triple impact energy level of 225 ft-kips at nearly the safe elastic limit. To extend the usual design energy level to smaller vessels of the 1,000-ton class or under and to express these energy levels in different practical units for convenient application, a schedule is established as shown in Table 1.

TABLE 1.—DESIGN AND ACCIDENT ENERGY LEVELS

Level of Impact Energy		Energy Units: Ft-Kips	Energy Units: In-Kips
(A) Design Level (at transient-load allowable working stresses)			
Ship Tonnage Class	1,000 or under	37 1/2	450
	2,000 to 20,000	75	900
(B) Accident Level (at nearly the safe elastic limit of materials)			
Ship Tonnage Class	1,000 or under	112 1/2	1,350
	2,000 to 20,000	225	2,700

BASIC RELATIONS OF ENERGY ABSORBING CAPACITIES

The energy absorbing capacity of a fender system is measured by the total amount of internal strain energy in flexure and shear; it can be safely set up and released. Three types of conventional fender-pile construction may arise as diagrammatically shown in Fig. 1. They are the (A) cantilever type, the (B) rigid-wharf type, and the (C) jetty type. The main internal strain energy in all these conventional types is that due to bending for long fender piles. But as shear always occurs simultaneously with moment by virtue of the relation $d\,M/dx = S$, its internal strain energy should not be neglected for short

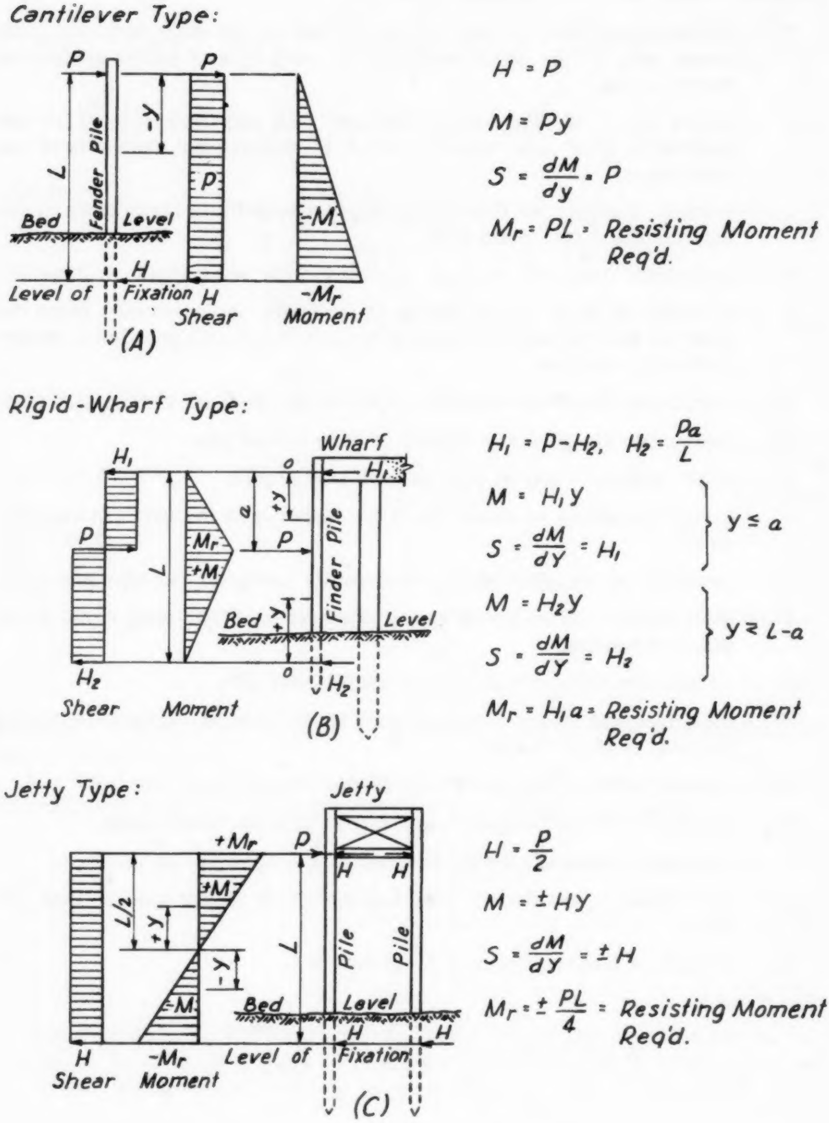

FIG. 1.—CONVENTIONAL TYPES OF FENDER-PILE CONSTRUCTION

piles and for wide-flange sections whose web area is limited, especially whenever higher accuracy is desired. Let

P = concentrated lateral load in kips acting on, or distributed to, each fender pile at the point assumed for each type of fender system as shown in Fig. 1.

L = length in ft of fender pile, between load and fixation level for the cantilever type, and between points of support for rigid-wharf and jetty types.

a = vertical distance in ft from top support to point of application of the load for the rigid-wharf type.

H = horizontal reaction in kips (denoted with subscripts in Type B).

y = vertical distance in ft measured upwards or downwards from the point of zero moment; positive, if associated with positive moment; contrary, negative.

M_r = maximum resisting moment required in kip-ft of each fender pile.

M = moment in kip-ft at any section of each fender pile.

S = shear in kips at any section of each fender pile.

E = Young's modulus of elasticity of the fender-pile material, in kips per sq ft.

G = modulus of rigidity of the fender-pile material, in kips per sq ft.

I = moment inertia in ft^4 of the cross section of the fender pile about plane of bending.

A = cross-sectional area in sq ft of each fender pile.

W = total internal strain energy of each fender pile as represented by its internal work in ft-kips.

W_f = internal work in ft-kips of each fender pile due to flexure.

W_s = internal work in ft-kips of each fender pile due to shearing.

f = maximum flexural stress of fender pile in kips per sq in.

s = simultaneous maximum shearing stress of fender pile in kips per sq in.

V = volume of fender pile in cu ft for length L.

For Type A,

$$W = W_f + W_s = \int_0^L \frac{M^2}{2\,E\,I}\,dy + \int_0^L \frac{S^2}{2\,G\,A}\,dy$$

$$= \int_0^L \frac{\left(\dfrac{M_r}{L}\,y\right)^2}{2\,E\,I}\,dy + \int_0^L \frac{P^2}{2\,G\,A}\,dy$$

$$= \frac{M_r^2\,L}{6\,E\,I} + \frac{P^2\,L}{2\,G\,A} = \frac{M_r^2\,L}{6\,E\,I} + 1\,\frac{P^2\,L}{2\,G\,A} \quad \ldots\ldots\ldots\ldots \quad (18)$$

For Type B,

$$W = \int_0^a \frac{\left(\frac{M_r}{a}\, y\right)^2}{2\,E\,I}\, dy + \int_0^{L-a} \frac{\left(\frac{M_r}{L-a}\, y\right)^2}{2\,E\,I}\, dy + \int_0^a \frac{H_1^2}{2\,G\,A}\, dy + \int_0^{L-a} \frac{H_2^2}{2\,G\,A}\, dy$$

$$= \frac{M_r^2\, L}{6\,E\,I} + \frac{P^2\, a\,(L-a)}{2\,G\,A\,L} = \frac{M_r^2\, L}{6\,E\,I} + \frac{a\,(L-a)}{L^2}\; \frac{P^2\, L}{2\,G\,A} \quad \ldots\ldots\ldots\ldots\ (19)$$

For Type C,

$$W = 2\int_0^{\frac{L}{2}} \frac{\left(\frac{2\,M_r}{L}\, y\right)^2}{2\,E\,I}\, dy + 2\int_0^{\frac{L}{2}} \frac{\left(\frac{P}{2}\right)^2}{2\,G\,A}\, dy$$

$$= \frac{M_r^2\, L}{6\,E\,I} + \frac{P^2\, L}{8\,G\,A} = \frac{M_r^2\, L}{6\,E\,I} + \frac{1}{4}\, \frac{P^2\, L}{2\,G\,A} \quad \ldots\ldots\ (20)$$

It is to be noted that $W_f = \frac{M_r^2\, L}{6\,E\,I}$ for all types; and by calling

$$\alpha = 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (21a)$$

for Type A

$$\beta = \frac{a\,(L-a)}{L^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (21b)$$

for Type B
and

$$\gamma = \frac{1}{4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (21c)$$

for Type C, the general strain energy equation may be written for all types as

$$W = W_f + \kappa\, W_s = \frac{M_r^2\, L}{6\,E\,I} + \frac{\kappa\, P^2\, L}{2\,G\,A} \quad \ldots\ldots\ldots\ (22)$$

in which $\kappa = \alpha$, β, or γ which are dimensionless parameters depending on the type of construction.

For Type B, in order to secure the maximum capacity of internal work due to shear, the value of "a" should preferably be arranged such that

$$\frac{d\beta}{da} = 0 = \frac{(L - 2\,a)}{L^2} \quad \ldots\ldots\ldots\ldots\ldots\ (23)$$

that is

$$a = \frac{L}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (24)$$

and

$$\beta = \frac{1}{4} = \gamma \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (25)$$

For wide-flange steel fender piles, when f is expressed in kips per sq in., and M_r, I, A, c (most remote fiber distance from the neutral axis, r (radius of gyration about x-axis), and A_w (area of web), in units of kips and ft,

$$M_r\, \frac{c}{I} = (12)^2\, f \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (26a)$$

and

$$M_r = (12)^2\, f\, \frac{A\, r^2}{c} \quad \ldots\ldots\ldots\ldots\ldots\ (26b)$$

and from Eqs. 16,

$$W = (12)^4 \left(\frac{f\,r}{c}\right)^2 \frac{A\,L}{6\,E} + \kappa\,P^2\,A_w\,\frac{L}{2\,G\,A_w^2} \quad \ldots\ldots\ldots \quad (27)$$

which reduces to

$$W = 3{,}460 \left(\frac{r}{c}\right)^2 \frac{V\,f^2}{E} + \kappa\,V_w\,\frac{\left(\frac{P}{A_w}\right)^2}{2\,G} \quad \ldots\ldots\ldots \quad (28)$$

For all wide-flange sections, both $(r/c)^2$ and A_w/A lie within narrow ranges. In the case of 12-in. wide-flange sections, $(r/c)^2$ has an average value of 0.718, and A_w/A an average value of 0.256. Taking these values,

$$W = V\,\frac{2{,}480}{E}\,f^2 + \frac{0.128\,\kappa}{G}\left(\frac{P}{A_w}\right)^2 \quad \ldots\ldots\ldots\ldots \quad (29)$$

For square fender piles of reinforced concrete, since $f_c = 2\,M_r/(j\,k\,b^3)$ in which $f_c = 1.2$ kips per sq in. for 3,000-psi concrete, $f_s = 20$ kips per sq in., $j = 0.875$, $k = 0.375$, $j\,k/2 = 0.164$; and if M_r and b are expressed in units of kips and ft, Eq. 30 is obtained as

$$M_r = 0.164\,(12)^2\,b^3\,f_c = 23.6\,b^3\,f_c \quad \ldots\ldots\ldots\ldots \quad (30)$$

and

$$W = \frac{12\left(23.6\,b^3\,f_c\right)^2\,L}{6\,E\,b^4} + \frac{\kappa\,P^2\,L\,b^2}{2\,G\,j^2\,b^4} \quad \ldots\ldots\ldots\ldots \quad (31)$$

which reduces to

$$W = V\left[\frac{1{,}120}{E}\,f_c^2 + \frac{\kappa}{1.53\,G}\left(\frac{P}{A}\right)^2\right] \quad \ldots\ldots\ldots\ldots \quad (32)$$

For square fender piles of timber, since $M_r\,b/(2\,I) = (12)^2\,f$,

$$M_r = 24\,b^3\,f$$

$$W = \frac{2\left(24\,b^3\,f\right)^2\,L}{E\,b^4} + \frac{\kappa\,A\,L}{2\,G}\left(\frac{P}{A}\right)^2 \quad \ldots\ldots\ldots\ldots \quad (34)$$

which reduces to

$$W = V\left[\frac{1{,}150}{E}\,f^2 + \frac{\kappa}{2\,G}\left(\frac{P}{A}\right)^2\right] \quad \ldots\ldots\ldots\ldots \quad (35)$$

Eqs. 29, 32, and 35 give respectively the total internal strain energy of one wide-flange-steel, square-reinforced-concrete, and square-timber, fender pile within the loaded vertical span (L). In each case, the first term represents the internal work due to flexure and the second term that due to shear. Only in Type A, and only when timber piles are used, P/A = average unit shearing stress; while in all other cases P/A is merely a factor of the internal work due to shear. The implications of these formulae throw the following guiding lights that have not been well recognized heretofore:

(1) In designing a conventional-type fender system to resist a certain impact energy appraised as kinetic energy delivered by a berthing ship, the most direct way is to provide the total internal strain energy (W) of the fender piles within allowable working stresses equal to or not less than the kinetic energy.

(2) The total internal strain energy is in whole directly proportional to the volume of material of fender pile within the vertical span (L), and in parts

proportional to f^2/E and $(P/A)^2/G$, in which A becomes A_w in wide-flange steel piles.

(3) The parameter κ is dimensionless and when $a = L/2$, $\kappa = \beta = \gamma$ for both Type B and Type C, while Type A is only for guide piles and ferry slips.

(4) The material that has the greatest internal-strain-energy capacity (W) within the vertical span (L) and suffers the least biological, physical, and chemical damages will have the greatest potentiality of being the most economical in capitalized or annual cost.

(5) The greatest internal-strain-energy capacity (W) requires greatest values of V, f^2/E, and $(P/A)^2/G$. They are not favorable in wide-flange steel piles for V is the least; least favorable in reinforced concrete piles for the ratio f^2/E is the least. Southern yellow pine and Douglas fir piles are more

TABLE 2.—MECHANICAL PROPERTIES OF STEEL, CONCRETE AND COMMON TIMBER

Material	Modulus of Elasticity		Modulus of Rigidity		Allowable Stresses		
	1,000 psi	1,000 k per sq ft	1,000 psi	1,000 k per sq ft	Extreme Fiber in Bending k per sq in	Transverse Shear k per sq in	Comp. Perp. to Grain k per sq in
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
ASTM A36 steel	29,000	4,180	11,200	1,610	20	12	
Reinforcement	29,000	4,180			20	$0.075f_c'$ on bjd	
Concrete (3,000 psi, n = 10)	2,900	418	1,320	190	1.2		
Douglas fir (coast region)	1,600	230	100	14.4	1.76[a]		0.185
Southern yellow pine (long leaf or short leaf)	1,600	230	100	14.4	1.76[a]		0.185

[a] For transient impact, this allowable value may be increased to 2.64 ksi.

favorable than reinforced concrete piles, as the ratio f^2/E is much larger for the same V. Greenheart fender piles have the greatest internal-strain-energy capacity, for they have all the required qualities.

These statements will become evident after we have assembled and derived pertinent mechanical properties of steel, reinforced concrete, and timber in the following section.

MECHANICAL PROPERTIES OF STEEL, REINFORCED CONCRETE, AND COMMON TIMBER

Selection of material according to its properties to best suit for its functional purpose and environmental condition is an essential part of an appropriate design. In Table 2 are assembled mechanical properties of ASTM-A36

steel, 3,000-psi concrete and intermediate reinforcement according to current standard design specifications. Moduli of rigidity of steel and concrete are derived from the equation,

$$G = \frac{E}{2(1 + \mu)} \qquad (36)$$

in which μ = Poisson's ratio = 0.3 for steel, and 0.1 for concrete. Allowable stresses in flexure and shear for the new ASTM A36 steel are each increased by 10% from the 18 ksi and 11 ksi used for ASTM A7 steel.

Test results of modulus of rigidity of timber in shear are meager. This modulus must be associated with shear deformation in one of the 3 mutually perpendicular planes defined by the L (longitudinal), T (tangential), and R (radial) directions and with shear stresses in the other 2 planes. Forest Products Laboratory of the U. S. Department of Agriculture recommends (12) approximate values of $G_{LT}/E_L = 0.06$, $G_{LR}/E_L = 0.075$, and $G_{RT}/E_L = 0.018$ for common species having no specifically tested values. For Douglas fir, southern yellow pine, and greenheart under average service conditions of partially submerged fender piles, their overall modulus of rigidity for shear perpendicular to grain may be taken in the order of $E_L/16$.

Mechanical properties of Douglas fir (coast region) and southern yellow pine (longleaf or shortleaf) are approximated (12) under average conditions with basic extreme fiber stress in bending at 2,000 psi, basic proportional limit of extreme fiber stress in bending at 5,200 psi, and basic compression perpendicular to grain at 210 psi, all reduced by 12% allowing for knots not over $1\frac{1}{2}$ in. in size, and without reduction for decay hazard or moisture condition as only treated material will be used. The reduced proportional limit of extreme fiber stress in bending is 4,580 psi. For triple-energy-level design and transient impact, the allowable extreme fiber stress in bending may be increased to 2.64 ksi, which is arrived at from $\sqrt{(4.58)^2/3}$.

MECHANICAL PROPERTIES OF GREENHEART

Mechanical properties of greenheart (Nectandra Rodioei Schomb., Ocotea Rodiei Mez) of Demerara, British Guiana, as tested by five different laboratories in the United States and the United Kingdom are compiled in Table 3. They are mostly tested in the form of small clear specimens and in the green or air-dry conditions.

As moisture content has a pronounced affect on the mechanical properties of timber and as greenheart will be used untreated, it becomes desirable to arrive at its allowable stresses at the re-saturated condition in service. The natural fiber-saturation point for most timbers is at approximately 30% moisture content at which the free water in the cell cavities has been evaporated and the cell walls are still saturated. On further air-drying, more moisture content will be lost. But after installed as fender piles, the untreated greenheart, which is naturally dense, will become re-saturated at a certain moisture content. For determining allowable stresses, this may be taken at 25% (though the actual moisture content under partially submerged conditions may be more), for toughness or shock resistance may also increase with further increase of moisture content.

Modulus of elasticity of greenheart under flexure versus moisture content is plotted in Fig. 2 which gives 3,200,000 psi as the modulus of elasticity at 25% moisture content. This elastic property appears as a denominator in the internal-strain-energy equation. For conservative reasons, it is not advisable to reduce this value for variation as it would then increase the energy-abosrbing capacity on the unsafe side.

Static-bending stress at proportional limit versus moisture content is plotted in Fig. 3 which gives 14,550 psi as the static-bending stress at pro-

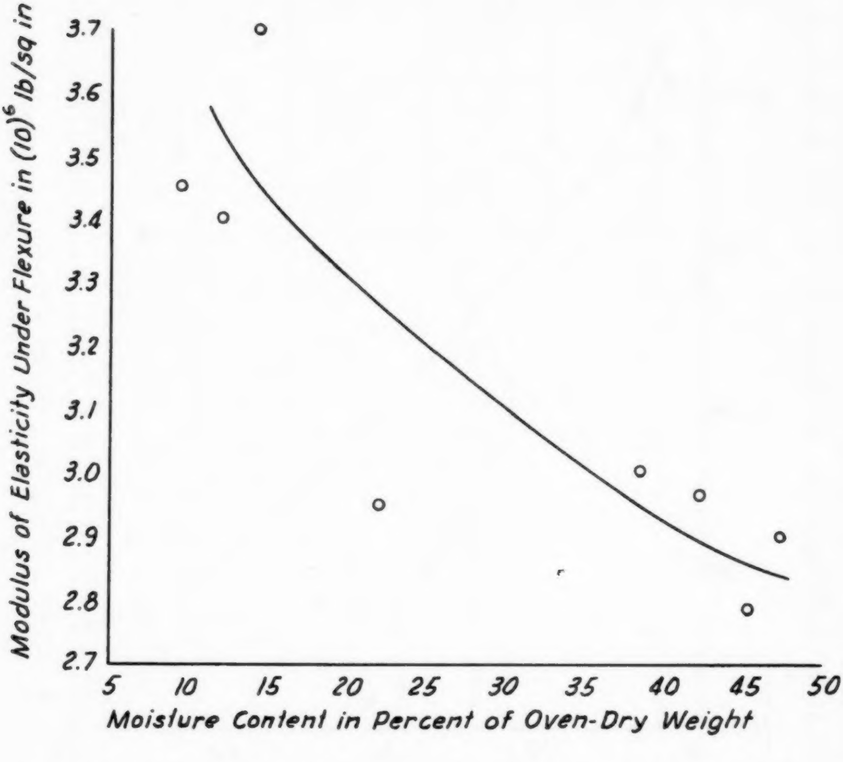

FIG. 2.—MODULUS OF ELASTICITY UNDER FLEXURE VERSUS MOISTURE CONTENT FOR DEMERARA GREENHEART (NECTANDRA RODIOEI SCHOMB)

portional limit corresponding to 25% moisture content. Allowing $17\frac{1}{2}\%$ for variations, a reliable proportional limit is found at 12,000 psi. Using a factor of safety of 1 2/3, an allowable flexural stress of 7,200 psi may be safely used for transient loading. Further reducing this value to 9/16, we get an allowable static-bending stress of 4,000 psi for long-continued loading. As fender piles are subjected only to impact loads of transient nature, for

triple-energy-level designs. a conservative allowable flexural stress of 6,900 psi is recommended. This is arrived at in the following way:

The accident-impact-energy level of 225 ft-kips is set at three times the design-impact-energy level of 75 ft-kips. As the major part of the internal strain energy is due to flexure, within proportional limit, other things being

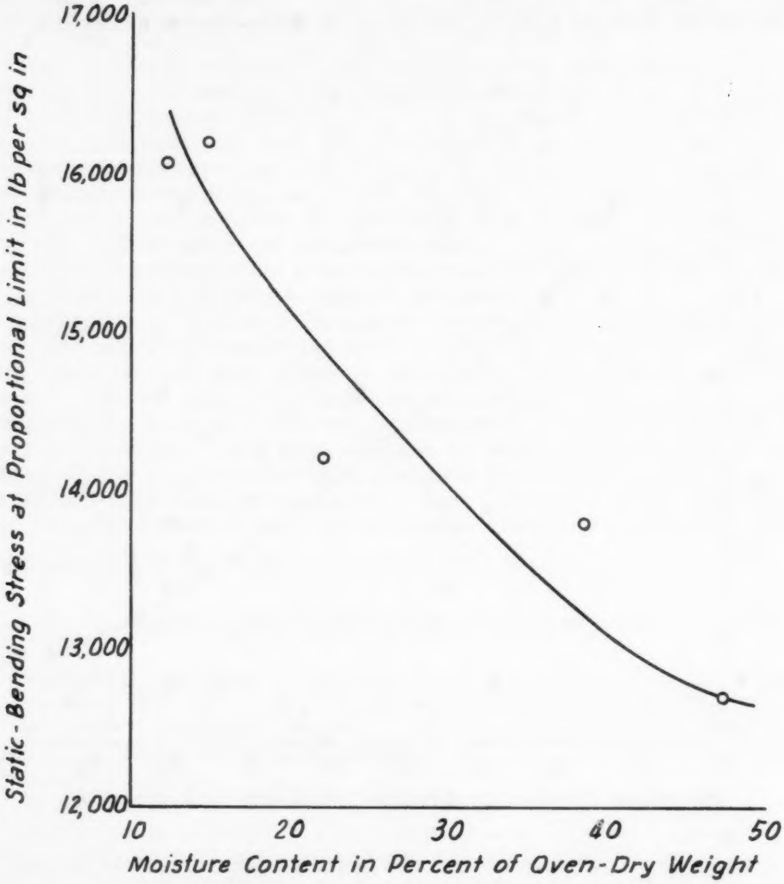

FIG. 3.—STATIC-BENDING STRESS AT PROPORTIONAL LIMIT VERSUS MOISTURE CONTENT FOR DEMERARA GREENHEART (NECTANDRA RODIOEI SCHOMB)

equal, the internal strain energy is proportional to the square of flexural stress. If greenheart fender piles are stressed in flexure at just the safe proportional limit of 12 kips per sq in. when subjected to accident-impact-energy level, the allowable working stress at design-impact-energy level must be not over $\sqrt{\frac{(12)^2}{3}}$ = 6.9 kips per sq in.

TABLE 3.—STRENGTH AND RELATED PROPERTIES OF GREENHEART (NECTANDRA RODIOE SCHOMB., OCOTERA RODIEI MEZ) OF DEMERARA, BRITISH GUIANA

Physical, Mechanical, and Elastic Properties (1)	Forest Products Lab., U.S. Dep't. of Agriculture, Pub. 1915 (2)		Columbia University Tests		Chesapeak and Ohio Ry.		University of Michigan Tests Published in 1938				Forest Products Research Laboratory United Kingdom (11)	
			1930 (3)	1931[a] (4)	Tests 1936 (5)	Estimated* (6)	(7)	Black Variety (8)	(9)	Black Variety (10)		
Seasoning	Green	Air-Dry		Air-Dry	Air-Dry		Green	Green	Air-Dry	Air-Dry	Green	Air-Dry
No. of Trees Tested				3			1	1	1	1		
No. of Test Specimens			n	n								
Moisture Content (% of Oven-Dry Wt.)	45.0	9.5	m	m	27.7	*12.0	47.1	38.3	15.1	14.4	42.0	12.0
Wt. per cu ft Green (lb)			w	w	59		78	78	65	66	76[d]	62
Static Bending			(Sub 1)	(Sub 3)								
Stress at Proportional Limit (psi)				14,200	9,910[c]	16,100	12,700	13,800		16,200		e{26,900
Modulus of Rupture (psi)	18,200	26,400	23,400	21,000	26,200	32,700	18,200	20,900		25,500	19,300	30,500}
Modulus of Elasticity (1000 psi)	2,790	3,450		2,950			2,900	3,040		3,700	2,970	e{2,970
Elastic Limit in Component ⊥												3,400}
to Grain (psi)	1,630	2,640									1,320	2,830
Compression Parallel to Grain			(Sub 1)	(Sub 4)								
Stress at Proportional Limit (psi)			8,180	10,100	5,820	9,460	7,060	8,100	9,870	10,140		{14,100
Maximum Crushing Strength (psi)	9,910	13,600	10,990	11,800	8,990	15,600	9,630	10,690	12,800	13,000	10,500	14,940}
Modulus of Elasticity (1000 psi)				3,570	3,860		3,470	3,700	4,170	4,140		
Compression Perpendicular				(Sub 4)								
to Grain							1,720	2,350	1,850	2,090	1,920	2,560[f]
Stress at Proportional Limit (psi)				2,570	1,510	3,550						
Ultimate Strength (psi)				4,940								
Hardness												
Indenting Load on Side Grain (lb)				2,090[b]	1,720	2,190	2,350	2,280	2,630	2,630	2,110	{2,520 / 2,650}
Shearing Parallel to Grain			(Sub 2)									
Maximum Shearing Strength (psi)	1,920	1,860	1,590		1,570	2,160	1,710	1,750		1,830	1,320	2,830
Tension Perpendicular to Grain												
Maximum Tensile Strength (psi)							1,000	1,140		1,020		

[a] Results of Static Bending tested flatwise not included. [b] 2 specimens, 4 readings each. [c] Erratically low, to be disregarded. [n_1 = 2, m_1 and w_1 not given; n_2 = 6, m_2 and w_2 not given. n_3 = 2, m_3 = 21.8 (av.), w_3 = 60.8 (av.); n_4 = 3, m_4 = 17.5 (av.), w_4 not given.] [d] At 50% moisture content. [e] Small-size test pieces. [f] Approximate calculated value.

Table 3 gives only 4 values of stress for greenheart at elastic limit in component perpendicular to grain under static bending. From these, an average of 2,110 psi is obtained corresponding to an average of 27.1% moisture content. Again by allowing $17\frac{1}{2}\%$ for variation and using a safety factor of $1\frac{2}{3}$, a transverse shearing unit stress is arrived at 1,040 psi which is raised to 1,100 psi as allowable for the assumed moisture content of 25%.

Compression perpendicular to grain of greenheart is useful for designing fender-pile bearing on wales. Its test values at proportional limit in Table 3 are too scattered; their average value is 2,240 psi corresponding to an average moisture content of 25.1%. By taking 2,250 psi as the value at 25% moisture content, allowing again $17\frac{1}{2}\%$ for variation and using a safety factor of $1\frac{2}{3}$, the allowable compression perpendicular to grain is arrived at 1,110 psi.

The mechanical properties of greenheart relevant to fender-pile design are summed up for re-saturation moisture content of 25% as follows:

	lb per sq in.	kips per sq ft
Modulus of Elasticity	3,200,000	461,000
Modulus of Rigidity	200,000	28,000
Safe Static-Bending Stress at Proportional Limit	12,000	
Allowable Bending Stress (long-continued loading)	4,000	
Allowable Bending Stress for Triple-Energy-Level Design (transient loading)	6,900	
Allowable Transverse Shearing Stress	1,100	
Allowable Compression Perpendicular to Grain	1,110	

SIMPLIFIED EQUATIONS AND RELATIVE MERITS OF DIFFERENT MATERIALS

To keep the form factor almost alike in evaluating relative merits of different materials for fender piling, consider wide-flange steel sections and square sections of reinforced concrete, Douglas fir, southern yellow pine, and greenheart according to already derived Eqs. 29, 32 and 35. By dropping, in these equations, the last terms representing internal strain energy due to transverse shear, which are, in general, less than 1% and rarely more than 2%; and by substituting the recommended values of moduli of elasticity and extreme fiber stresses in bending, simplified equations and comparative results of internal-strain-energy capacity per fender pile for different materials are obtained, as shown in Table 4 and further extended in Table 5.

An examination of these tables will at once reveal the following facts:

(1) Fender piles of the same material but of different sizes, such as shown for steel and greenheart, have internal-strain-energy capacities almost in direct proportion to their volumes or cross sections for equal vertical length between supports.

(2) Reinforced-concrete fender piles of the same overall dimensions as steel and timber have the least internal-strain-energy capacity. Either their

TABLE 4.—SIMPLIFIED EQUATIONS AND COMPARATIVE RESULTS OF INTERNAL-STRAIN-ENERGY (W) CAPACITY PER FENDER PILE

Material (1)	Expression of W per Pile (V = AL) (2)	Size L = 50 ft (3)	W per Pile (ft-kips) (4)
ASTM A36 steel	$240V/(10)^3$	12 WF 190 12 WF 65	4.69 1.61
Reinforced concrete	$386V/(10)^5$	12 x 12	0.194
Douglas fir or southern yellow pine	$348V/(10)^4$	12 x 12	1.75
Greenheart At proportional limit Allowable	$359V/(10)^3$ $119V/(10)^3$ (Simplified Equations)	12 x 12 12 x 12 8 x 8 6 x 6	18.00 6.00 2.65 1.49

TABLE 5.—COMPARATIVE MERITS IN ENERGY-ABSORBING CAPACITY L = 50 FT. TYPES B AND C

Material and Size of Fender Pile	Internal Strain Energy Capacity at Allowable Working Stress (Ft-Kips)	Energy Capacity Versus Greenheart of Same Overall Size	No. of Fender Piles to Resist 75 Ft-Kips	Spacing of Fender Piles for Contact Length of 150 Ft	Remarks
12 WF 190	4.69	$\frac{1}{1.3}$	16	10 ft-0 in.	
12 WF 65	1.61	$\frac{1}{3.7}$	47	3 ft-3 in.	
12 x 12 R.C.	0.194	$\frac{1}{3.1}$	387		Not Suitable
12 x 12 Douglas Fir or Southern Yellow Pine	1.75	$\frac{1}{3.4}$	43	3 ft-9 in.	
Greenheart At f = 12 k per sq in. 12 x 12	18.0	3	13	12 ft-6 in.	For 225 ft-kips impact energy level
Greenheart At f = 6.9 k per sq in. 12 x 12	6.00	1	13	12 ft-6 in.	
Greenheart At f = 6.9 k per sq in. 8 x 8	2.65		28	5 ft-6 in.	
Greenheart At f = 6.9 k per sq in. 6 x 6	1.49		50	3 ft-0 in.	

spacing may be unsuitably crowding or their size would be too big. They are too rigid for the required resilience, may cause fear for bumping, and result in casting off. Furthermore, they do not have a well defined proportional limit and hence are unsuitable for the triple-energy-level design to provide a high accident-impact-energy limit.

(3) Greenheart fender piles of the same overall dimensions as other materials have 1.3 times energy-absorbing capacity as the heaviest 12 WF steel sections, 3.7 times as the lightest 12 WF steel sections, 31 times as reinforced concrete, 3.4 times as Douglas fir and southern yellow pine.

(4) Steel-wide-flange sections, though high in allowable flexural stress, are low in volume, and hence low in energy-absorbing capacity. On the other hand, reinforced concrete, though high in volume, is low in concrete flexural stress, and results in low energy-absorbing capacity. When it has sufficient energy-absorbing capacity, it will damage berthing ships coming with high kinetic energy.

(5) Greenheart has a well defined proportional limit; it takes design-impact energy at 6,900 psi and triple-design-impact energy at its safe proportional limit of 12,000 psi in the examples computed for Tables 4 and 5. This provides a wide range to avoid any, except unexpected, damages.

(6) Internal strain energy due to shear may be neglected in all cases of three-figure accuracy, though it becomes significant in wide-flange steel sections. Those who carry to four or more significant figures should not neglect this energy for reason of consistency rather than absolute necessity.

(7) Table 5 has been computed for a 2,000-ton class ship with only 150-ft parallel wall side. Fender piles, if adequate for this length of contact, will be safe for larger ships with longer contact lengths, approaching at lower velocities and not with full mass acting.

WORKING FORMULAE AND EXAMPLE FOR ROUND FENDER PILES

For round fender piles of treated timber or untreated greenheart of average diameter d ft (for convenience of writing equations), and vertical span L ft, by dropping the second term of Eq. 22 for internal strain energy due to shear,

$$W = \frac{M_r^2\ L}{6\ E\ I} \qquad (37)$$

But

$$M_r = \frac{144\ f\ I}{c} = \frac{288\ f\ I}{d} \qquad (38)$$

and

$$I = \frac{d^4}{64} \qquad (39)$$

Therefore

$$W = \frac{(288)^2\ \pi\ d^2\ L}{6\ (16)\ 4\ E}\ f^2 = \frac{864\ V\ f^2}{E} \qquad (40)$$

For Douglas fir and southern pine with E = 230,000 ksf, and f = 2.64 ksi for transient impact and triple-energy-level design,

$$W = 0.0262\ V \qquad (41)$$

For greenheart with E = 461,000 ksf, and f = 6.90 ksi for transient impact and triple-energy-level design,

$$W = 0.0892\ V \qquad (42)$$

in which W is in ft-kips, and V in cu ft.

To illustrate the simplicity in the application of Eqs. 41 and 42, let the impact energy be 75 ft-kips, vertical span of fender piles between supports 50 ft, and available average diameter of piles 12 in., the internal-strain-energy capacity of each pile is given, for greenheart, by

$$W = 0.0892\ (50)\ \pi\ \frac{(1)^2}{4} = 3.50 \text{ ft-kips}$$

$$\text{No. of piles required} = \frac{75}{3.5} = 22$$

If the shortest class of ship that may berth alongside has a parallel wall side of 147 ft, then

$$\text{Spacing of fender piles} = \frac{147}{(22-1)} = 7 \text{ ft.}$$

CONCLUSIONS

This paper has demonstrated, in particular, the development of the operative energy concept in marine fendering, the soundness of its basis and application, the appraisal of impact loads and impact energy that may be delivered by berthing ships, the evaluation of internal-strain-energy capacity of fender piles, the determination of mechanical properties of greenheart and the correlation of them relevant to triple-energy-level fendering design, the relative demerits of other materials versus greenheart when used in fender piling, the simplicity of the working formulae and their ease of application.

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ECONOMIC EVALUATION OF INLAND WATERWAY PROJECTS

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SYNOPSIS

Basic data requirements for the economic evaluation of inland waterway projects are summarized and some of the current evaluation practices are given. To facilitate further study of the subject, an extensive chronological bibliography pertaining to the economic evaluation of waterway projects is included.

INTRODUCTION

It has been said that the study of navigation economics involves much research into the prevailing pattern of commodity production and marketing, the factors controlling distribution, and the past, present, and indicated future economic activity of the entire region. The basic objective being to determine from the public viewpoint whether the transportation service provided by improvement for navigation will be beneficial. The service is considered beneficial if it results in transportation at less total expense than that of equivalent transportation in the absence of the project. Four principal types of benefit are considered in the economic evaluation: improvement of operating conditions for existing navigation; provision of more economical transportation; stimulation of new commerce; and economic and social improvements. This paper briefly

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presents the writers' views on these basic items and suggests that construction costs now assigned to navigation improvements inherently include those which should be assigned to public conservation of the water course; no part of the paper is to be construed as official statements or opinions of any governmental or other body with which the writers now are or may have been connected except as may be specifically cited herein.

CURRENT PRACTICES

Many agencies and organizations have functions of varying importance in the development of the nation's rivers and harbors, including inland waterways and the Great Lakes. Some of the groups prepares original and complete studies on cost-benefit evaluation for the improvement and extension of various waterways. Each of these groups have their methods and practices which they use in determining to their satisfaction the economic feasibility of the improvement considered.

Studies favorable for development and improvement demonstrate by various means the ability to provide more efficient and economical transportation in comparison with alternative modes of transport. Such studies usually begin with an accurate determination of the economic region tributary to the waterway project. Consideration is given to the evaluation of all effects of a project, tangible and intangible. No one has, however, a specific "formula" for estimating the benefits of a waterway improvement. Such benefits are appraised on the basis of (1) detailed studies of traffic that may be expected to use the waterway and (2) transportation savings that will accrue to the public. Generally these savings are computed as the difference between the transportation charges to the shipper or consignee of shipping over the waterway and that over the least expensive alternative. Currently, consideration is being given by transportation economists to determining navigation benefits on the comparison of long-run marginal cost of providing the service. Rate comparisons, nevertheless, would still be necessary to appraise the attractiveness of the waterway to the shipping public in order to estimate the prospective tonnage which would be attracted to, or generated by, the less expensive water transport; the use of the long-run marginal cost of providing the service would indicate the economic comparison.

Thus the writers believe that future recommendations for improvements to inland waterways might be based on the comparison of both rates and costs: the projected use of the waterway would be estimated on the basis of rates charged for its use versus those charged for transport by other media or alternate routes; and the over-all economic gain to the nation would be measured by the savings in the long-run marginal costs of transportation over the waterway as compared to the long-run marginal costs over other media or alternate routes.

Although the transportation saving contributes to the general economic benefit that will inure to the nation as a whole, the use of a waterway does not depend on it but rather on the immediate benefits that may be derived by those to whom the improvement will be directly available. It seems that the volume of commerce that would be moved over a waterway is not induced by what it costs the carrier to transport it, but rather by what it costs the shipper or con-

signee to have the freight moved from origin to destination. To perform the analyses which determines the value of a waterway project certain basic data are required.

BASIC DATA REQUIREMENTS

In order to determine the economic justification of an inland waterway improvement, a great deal of basic data must be collected. Statistics are required on existing commerce, equipment used, population and production in the region, natural resources, carrier operational costs, freight rates, etc. Some of these data, particularly those pertaining to operation costs for bargelines, and trucklines, are difficult to secure because of their confidential nature. Usually governmental regulatory authorities and commissions have much useful data in their files and public reports. With all the data available, the evaluation of navigation benefits is dependent, nonetheless, on the ability, knowledge, experience, and understanding of engineers, economists, and their assistants engaged in the techniques of collecting, as well as interpreting and analyzing the essential traffic data relevant to the different media of transport.

Water Transportation.—Data are required on the present commerce, the direction of movements, type of commerce, the trips and drafts of vessels using the project (actual draft of water craft, not the designed maximum or minimum draft). Point-to-point movement by commodity is required because line-haul costs are reflected therein and the movements are compared on an equivalent basis with the costs or charges of alternate forms of transport. To obtain the necessary data when a new waterway improvement is under investigation, a comprehensive field survey of the area expected to be affected by the improvement is made in order to obtain the information. In the United States, the volumes of past and present commerce, individual commodity movements, and the types, sizes, drafts, and trips of vessels moving over inland waterways and through ports are given in the annual reports of waterborne commerce published by the Corps of Engineers, U. S. Army. Data are also collected and published on ports and terminal facilities, and the nature and amounts of commerce handled through individual ports or at individual port-handling facilities. The Maritime Administration of the U. S. Department of Commerce issue statistical information on domestic oceanborne Great Lakes commerce of the United States. The Bureau of the Census of the same department issues statistics on the foreign waterborne commerce of the Great Lakes area. Further statistical data as well as descriptive material on shipping activities on the Great Lakes are given in the annual report of the Lakes Carriers' Association.

Rail Transportation.—Transportation statistics for railway freight movements are required wherever it is possible for a navigation project to serve a territory which is, or can be, served by railroads, and if it is found that rail transport is the next lowest expensive carrier for the transport of freight. Whenever possible, point-to-point movement by commodity is secured for the comparative analysis of transportation via the considered water route. Data on operating revenue and expenses, mileage, ton-miles, and information for the various railroads are published annually by the Bureau of Transport Economics and Statistics of the Interstate Commerce Commission.

Motor-Truck Transportation.—Transportation statistics for motor-truck freight movements are needed whenever truck competition with waterway move-

ment is evident and if analysis shows that cost of motor-truck movement is the next lowest in expense when compared with waterway movement. Specific motor-truck transportation statistics, particularly point-to-point movements, are difficult, if not impossible, to secure from any central source. Effort is made to obtain locally available data pertaining to motor-truck movements within the region in which the economic justification for an inland waterway improvement is under consideration. General data on operating revenue and expenses, ton-miles, and investment for the regulated carriers are published annually by the Bureau of Transport Economics and Statistics of the Interstate Commerce Commission.

Pipeline Transportation.—Pipeline transportation statistics are secured whenever it is apparent that movement by pipeline is in actual or potential competition with the inland waterway route under investigation. The large volume of fluid commodities which can be handled by pipelines gives them a favorable position to compete with inland waterway improvements, and in some cases existing pipeline transportation rates and the total through charges from point-to-point are lower than what the considered inland waterway could provide. This is particularly true in the case of "big-inch" lines operating between producing centers and refineries and between refineries and large consuming centers. Frequently, however, the shipper cannot meet the minimum tender requirements of the pipeline company, and he finds it economical to use tank-barge transportation on the inland waterways.

While volume of movement may dictate the choice on the one hand, capital investment may reverse the case on the other. A large refinery on the Ohio River found it more economical to invest in a fleet of towboats and barges rather than a pipeline. This company has a large-volume movement of crude oil, on regular schedule, from a port on the lower Mississippi River to its refinery, a situation ideally suited to pipeline movement. Nevertheless, the company found that a much smaller capital investment in a fleet of towboats and barges proved more economical than the construction of a pipeline, despite the large volume of movement involved. The experience of this company indicates first, the necessity for a thorough investigation and development of transportation information and statistics, even though it might appear obvious that pipeline movement would be more economical than barge movement, and second, that there may be other factors which in the long run would favor the inland waterway improvement under investigation. Data on operating revenues and expenses, mileage, barrel-miles, and investment are issued annually by the Bureau of Transport Economics and Statistics of the Interstate Commerce Commission.

Vessel-Types - Barges.—The appraisal of the need and value of any proposed navigation improvement involves a thorough analysis of the various types, sizes, drafts, barge formations, and operating characteristics of the vessels presently using or potentially tending to use the inland waterway under consideration. Usually two questions may arise for determination: (1) whether the existing facilities are adequate for the accommodation of anticipated future barge movements, and (2) whether in view of traffic trends and changes in design and size of barges the considered inland waterway improvement is justifiable in the interest of present and future commerce. Information relating to operating performance and expense of operation, as a rule, can only be obtained by special investigation or inquiry from individual barge operators. Inference from operating records can give information on the horsepower and speed of the towboat required to move the desired number and formation of barges over the proposed improvement.

Production.—The essential estimate of the prospective waterway traffic in specific commodities should be preceded by a comprehensive analysis of production, supply, and demand, in order to provide the basis for establishing the total tonnages that may be accepted as contributary sources of the prospective movements. Only through results of such analyses can the most dependable tonnages be selected for estimating the probable transportation savings creditable to the projected improvement.

The production of a specific commodity in the zone tributary to the projected waterway improvement may be either greater or less than the local demand. Thus, information on production and consumption is useful in judging the extent to which tonnages are available for movement into or out of, as well as within and through the reach of the waterway. Data in the following paragraphs indicate the nature of the basic information helpful in establishing a basis for judging the probable volume of traffic to be considered in estimating the prospective traffic.

1. Agricultural.—In an agricultural producing region, a projection for a period corresponding to the amortized life of the waterway improvement can be made of farm production and of sales, the sales being the potential source of traffic. It is important to distinguish between production and sales, as in some regions only a small percentage of production is sold while in others the reverse is true.

Industrial requirements for agricultural products in the region when measured against corresponding farm sales; ascertain the extent to which these requirements can be supplied by those sales. Cognizance should be taken of any preference shown for products from other regions even though there is a local supply. The extent to which any local supply runs short of local demand is usually indicative of the prospective inbound tonnage, and, conversely, any excess of local supply over demand constitutes a source of prospective outbound traffic.

An agricultural region usually requires a supply of fertilizers which constitutes another source of prospective inbound traffic. While available statistics indicate that but little mixed fertilizers move by barge, the separate ingredients do move long distances thereby. Local plants prepare the mixture suitable for the region. Principal ingredients are those which supply nitrogen, phosphorous, and potash, and each of these general categories may consist of one or more materials.

2. Petroleum.—The supply and demand criteria previously cited apply to the determination of whether the region would receive or ship petroleum products. Both highway and off-highway uses should be considered. To estimate the potential tonnage for highway use in the region, probable motor vehicle registration through the use of projected population and the number of persons per vehicle considered can be used as a basis. In establishing future vehicle registrations the analysis is guided by past registration in relation to population and by changing technological conditions. Non-highway uses include those for farm implements, railway diesels, motor vessels, aviation, construction, industry, commerce, and domestic consumption. It has been found that the estimate of non-highway use of petroleum products usually can be based satisfactorily on past relationships between highway and non-highway usage with recognition given to trends.

3. Forest.—In an undeveloped area, the prime commodities for outbound barge traffic may be the products of forests. The same general type of ana-

lysis based on supply and demand, extent of the resource, and projected trends are applied to determine the volume of timber which can be considered as prospective commerce. The timber from the forest area would be compared with other sources of similar material which could serve the same consuming center. Ownership of the supply may determine whether or not the product will move to consuming centers other than those owned or controlled by the same interest. If the prospective movement crosses a national boundary line or be for export, the movement may depend on existing international relations.

4. Mineral.—The basic information required for the development of an inland waterway to service a mining region involves the determination of reserves, prospective discovery, production, rate of depletion, and quality of products. If an undeveloped region has only mineral resources, its tapping by an inland waterway would form an integral part of the development. All costs probably would be amortized over a selected period of time but in no event longer than the expected life of the resources. The inland waterway in such an instance, though the resource has a limited life, might have a decisive influence on the development. Each situation has to be judged on its own merits and the overall economic justification determined accordingly.

Industrial Requirements.—The information required for the provision of a waterway to serve a new industrial complex or an expanding industrial district includes the classes, quantities, and sources of raw materials needed to supply the plants; the products, quantities, and their marketing centers; and in fact, all the facts that are normally considered for plant locations with respect to in and outbound transportation. An inland waterway barge service would be one of the important factors. Where giant heavy industries are located beyond the head of present sea-vessel navigation, upstream extension has come to the fore, such as the improvement of the Delaware River for navigation from Philadelphia, Pa. to Trenton, N. J.

Transport Media Available.—Basic data on transport media available in the region of the projected waterway include an inventory covering all types of transportation, their lines and mileage, together with complete description of their capacities, dependability, and costs of operation. It is important to note which will compete with, and which will be tributary to, the projected waterway.

Rail-Carrier Operating Costs.—The term costs as used herein refer to long-run marginal cost, not the charges to the shipper or consignee. Such rail costs have not been available and so rail rates are usually used in comparison with computed or actual water-carrier rates.

Pipeline Operating Costs.—These costs can be determined by an engineering study of the construction and operating cost characteristics. Rates are published by some lines and are suitable for comparison with rates by other transport media.

Trucking Costs.—For-rent charges or rates from representative truck companies are suitable for comparison with rates by other media. Actual trucking costs or probable rate for movement of a commodity can be determined to a reasonable degree by computing the expense for a hypothetical operation. If the required terminal handling expenses are not available for the operation of private or public terminals, an equipment and time study will reveal fairly representative figures. These expenses are an essential part of providing complete service by water carrier for comparison with other transport media as the major portion of inland waterway traffic neither originates nor terminates directly at waterside. The terminal facilities should be modeled to fit the needs of the shippers and consignees.

Terminal Handling.—Inland waterway terminals in the United States are of three general types with differently adapted handling:

1. Public, through which both general merchandise and bulk traffic are handled.
2. Semi-public, providing limited service for transfer of certain classes of freight usually in specific volume.
3. Private, limited to the handling of basic commodities needed by owner-industries.

The infinite variety of size, capacity, equipment, and uses to which many terminals are placed necessarily involves a wide variety of rate or charges. Computed terminal charges should include every item of outlay which the operator must recover in order to conduct the business on a satisfactory basis. Reasonable depreciation charges, taxes for general functions of governing bodies, adequate insurance, and a return on the investment sufficient to attract capital should be included. The computed rates to be useful must reflect seasonal operations, based on actual rather than theoretically attainable conditions, that is, on volume of traffic and a load factor that may likely develop over a period of years.

BENEFIT EVALUATION

The guiding principle in benefit evaluation is to handle each problem on its own merits and apply the most logical and realistic procedures as the particular circumstances may demand.

Value of Service.—Little commerce will move by water-carriers when it can move by other means at equal costs to the shipper or to the consignee. Therefore lower charges are necessary to make water transportation attractive to commerce moved by other modes of transport. The differences in charges may be regarded as a "value of service." This factor of value of service can generally be taken into consideration in the process of eliminating items of prospective waterway tonnage which show a smaller saving per ton than the amount estimated as necessary to attract the commodity to water transportation. The value of service factor is usually recognized in the elimination of tonnages of barge-adapted tonnages in recognition of the preference of shippers for services of those land carriers with whom they have close financial and commercial ralationship.

Interest Rates and Tax Factors.—Both land and water carrier charges include, in general, items to cover necessary taxes and interest at commercial rates. Annual carrying charges for federally provided inland waterway improvement are based on a lower prescribed federal interest rate and do not include taxes. Thus, the comparison of annual benefits, computed as the difference between charges by land and water carrier, with the annual costs for the waterway improvement may seem to produce a questionable ratio. As an attempt to reach a fair economic comparison with private transportation media, use of a non-federal interest rate and the full normal tax at a rate experienced by tax paying owners, was recommended by the Presidential Advisory Committee on Water Resources Policy in 1955.

It is considered doubtful if inclusion of taxes would bring about any substantial change in the competitive relationship between land and water carriers when all aspects involved in each project are considered. In addition to purely

navigational benefits, it is believed, that the value to states, local communities, private interests, and the general public, of inland waterway improvement, brought about by expenditures charged to navigation, usually far outweighs the proportionate revenue from any taxes that might be assessed on, or hypothetically assigned. These economic and social benefits created by waterway improvements, over and above the purely navigational benefits, if evaluable in monetary terms, might increase the monetary benefits of the improvements.

The federal annual charges computed for an inland waterway improvement include, beside operation and maintenance, interest and amortization on the investment over the project life, a period usually taken at 50 yr, both at the prescribed federal interest rate. The interest rate used for the non-federal portion of the project costs including amortization, are usually based on the average long-term cost of money in the project area.

Adverse Effects.—All effects from provision of a navigation improvement are, in general, evaluated and included in the economic analysis in order to determine the true worth of the proposed project. Sometimes insufficient recognition is given or allowance made for predictable adverse effects or "induced costs" of the proposed improvement on other means of transport. The net loss, if any, due to displacement of competitive transportation by diversion of traffic to water routes is sometimes given consideration in determining the net benefits available for project justification. In populated areas, account is taken, in the economic analysis of the harmful effect on the flow of overland traffic by delays and obstructions created as a result of bridge clearance and operational requirements for navigation.

Closed Season.—Generally the deficiency of the closed navigation seasons, such as on the St. Lawrence Seaway and the Great Lakes, is considered to be offset by the practice of shipping during the period of open navigation large tonnages of commodities in excess of immediate requirements, and stockpiling the excess for future needs. Most waterways have the ability to absorb a large expansion in traffic volume with little or no increase in cost, an ability not possessed by some of the other modes of transport. Occasionally other modes of transport will increase their charges during closed navigation seasons, not because of increased cost, but because of lack of competition.

Potentialities are recognized in the evaluation of navigation benefits. They include the increasing population, and enhancement in the standard of living, or at least the maintenance of the present high standards, which creates an increasing demand for new and better products, and in turn a parallel progress in the economic methods of production, marketing, and distribution. The fact that water transportation has proved in these activities to be the least expensive mode of heavy transport, presents an ever increasing demand for dispersement of industries along the waterways and an accompanying growth in waterborne commerce. The significance of these factors is translated into long-range predictions of growth of waterborne commerce.

A broad perspective of the potentialities of water transportation is needed when determining the economic justification of waterway improvements. Water transport has been, and is expected to continue to be, progressive, in order to meet future requirements of the public for an efficient, cheap transportation service which will arise from an ever increasing population with increased purchasing power. It is probable that in the years ahead waterways will assume an ever more important role in the transportation system of all countries.

Incremental Savings.—When improvement of existing navigation facilities is being evaluated, the effect such improvement might have on equipment oper-

ating costs is usually considered. When savings in operating time is the basis for benefit evaluation, it is the major savings in time that are usually taken into consideration. Minor savings in time are of questionable value and are usually disregarded unless available data fully substantiate the reality of the savings. If the navigation improvement would permit light-loaded barges to carry full loads and thereby accomplish the transportation of the commerce in fewer barges or trips, the saving involved is credited to the improvement.

Induced Costs.—It appears that because suitable techniques have not been developed, little more than lip service is made for predictable adverse effect or "induced costs" of proposed improvements on adjacent waterways and other transport media. The purpose of providing inland waterway improvements is to create new or improved navigation facilities from which benefits not presently available to the shipping public may be derived. Justification for inclusion of benefits from the provision of additional or improved facilities should include an exhibit of net benefit expected to be realized from the proposed facilities over that presently being secured or available by use of existing facilities. It is important to recognize in this connection not only the adverse effect by uneconomic duplication or displacement of navigation facilities on adjacent waterways, but also the deleterious effect that may result to regional transportation from the indiscriminate development of unneeded and competing waterways. The net loss, if any, due to displacement of facilities of competitive forms of transportation by the diversion of traffic to water routes should be given adequate consideration in determining the net benefits available for project justification.

Benefit evaluation practice should also recognize the deleterious effect on the flow of overland traffic by the delays and obstructions created as a result of bridge clearances and operation requirements for navigation, as well as the cost incurred in providing bridge clearances required now and in the future for navigation.

These various induced effects or costs are sometimes termed indirect costs or negative benefits. Monetary costs should be included for induced additional expenditures by local interests in capital equipment, labor, operation, and maintenance as negative benefits in computing the net benefits.

To trace further negative benefits or induced costs, the following may be appropriately enumerated:

1. Increased overland transportation cost per ton-mile due to decreased volume caused by diversion of traffic to water carrier.
2. Taxation loss on existing overland carriers or competing facilities due to revenue loss.
3. Taxation loss on security holders due to reduced dividends as a result of reduction in net revenue of overland carriers.
4. Public loss due to rate increase by existing carriers for operation and maintenance as a result of decreased traffic.

Benefits of all types resulting from an inland waterway improvement are usually evaluated and included in the economic analysis in order to provide a complete and realistic presentation of the worth of the proposed improvement.

COST ALLOCATION

Both the allocation of first cost and the allocation of annual cost require equitable consideration. Even when not a part of a multiple-purpose project

involving power development, and/or flood control, etc., the provision of an inland waterway, whether by channel improvement only, by canalization of existing rivers, or by building new canals, usually embraces, to a greater or less extent, conservation of the natural regimen. Associated with properly constructed and maintained inland waterways are bed and bank stabilization, minimization of erosion, deposition, and sediment transportation, and in some measure also flood relief and runoff equalization, all contributing to the strengthening of the water resource base on which the economy of a nation rests. If conservation so derived were to be executed in any event, then the first and annual costs chargeable thereto should be deducted from inland waterway costs. The equitable share of improvement cost and annual cost chargeable to inland waterways should be the remainder after all other cost allocations have been properly made.

BENEFIT-COST RATIO

With annual benefit evaluated and annual cost allocated including all tangible benefits and costs and with due consideration of all intangible benefits and costs, the principle that the benefit per cost ratio should be greater than one, has been regarded and used as a measuring stick for the economic justification of inland waterway projects. If two or more alternative projects for achieving the same result have been studied, then the project scheme which results in the highest benefit-cost ratio should be the most economical, provided each has been estimated on the same uniform basis and no intangible benefit-cost consideration could outweigh the tangible benefit-cost ratio. It also implies that an equivalent benefit would be impossible to achieve at a lower cost.

The latest documents put forth in support of the preceding principle are (1) "Report by the (U. S.) Presidential Advisory Committee on Water Resources Policy," December 22, 1955; (2) "Integrated River Basin Development," Report by a Panel of Experts, United Nations Department of Economic and Social Affairs, November 23, 1957; (3) "Cost Allocations for Multiple Purpose Projects," an Engineering Manual of the Corps of Engineers, U. S. Army, January 1, 1958; and (4) "Proposed Practices for Economic Analyses of River Basin Projects," a report to Inter-Agency Committee on Water Resources by Subcommittee on Evaluation Standards, May, 1958.

The benefit-cost ratio has been best known for simplicity and clarity, and has been applied in both the United States and abroad for lack of more refined yardsticks.

Normally, an inland waterway project should not be undertaken unless the benefit-cost ratio is greater than one. However, if intangible benefits or costs should prove to be of substantial significance, the ratio must be weighed against such intangible values in the final decision.

In comparing the benefit-cost ratios among alternative projects that would achieve the same results as an inland waterway, not only alternative waterway schemes but also railway and highway plans should be considered if the purpose is for general freight transportation; and further, pipe lines and belt conveyors should also be considered if appropriate and if the intended transportation is for special bulk fluid or solid commodity. In such cases the project that gives the greatest benefit-cost ratio is the most economical, although not necessarily the most desirable.

Within the inland waterway project itself, the inclusion or omission of any feature, such as cut-off or straightening, the extent of fairway width and depth, the height and location of dams, the dimensions of locks, and the extent of navigation aids, etc., should be weighed against present and future needs, and in addition, justified by the benefit-cost ratio for each separate feature such that no feature should be over or under projected beyond the point where the benefits added for the last increment are equal to the cost of its inclusion, or the benefits decreased for the last decrement equal to the saving of its omission.

APPENDIX.—SELECTED BIBLIOGRAPHY[3]

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STEEL PILE MARINE CORROSION AND CATHODIC PROTECTION[a]

By Carter Hale Horton,[1]

SYNOPSIS

Corrosion which occurs on marine structures, and the use of cathodic protection to prevent such corrosion are presented herein. Briefly, the corrosion studied in this paper is an electrochemical action on steel which occurs only in the presence of water, and results in deterioration of the steel. Cathodic protection is an electrical method of preventing this corrosion.

This paper is directed toward the civil engineer who is concerned with structures in corrosive environments. The position of corrosion zones (earth, water, tidal, and splash) on vertical steel members is analyzed coupled with the range of corrosive attack normally encountered.

The use of thicker metal sections is presented as well as coverings of various types. The principles of cathodic protection are given for galvanic and impressed current systems. The various ways of installing anodes are described as well as the affect on the cathodic protection electrical current. How fresh, brackish, and salt water affect the current flow is also included.

Construction considerations show how steel members can be made metallically continuous so that cathodic protection may be applied. Utilities in the area must also be considered. Concrete capping and its extent of coverage is shown to affect the corrosion rate. The electrical considerations for cathodic protection, test points and ship bonding are noted for hazardous locations.

Note.—Discussion open until January 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 3, August, 1961.

[a] Presented at the Annual ASCE Convention at Boston, Mass., in October, 1960.

[1] The Hinchman Corp., Detroit, Mich.

Some ideas for protecting anodes and cable are given. Why corrosion engineering should enter into the project in the planning stage is presented.

WATERFRONT STRUCTURES INVOLVED

The corrosion and its prevention, as studied herein, applies to wharves, bridges, and similar structures in which steel members are in contact with water or soil or both. Usually these steel members are vertical and are driven into the earth with the upper part contacting water. The members might be sheet piles in a quay wall on perhaps "H" or cylindrical piles in a pier.

Equipment used in construction, such as floating cranes, pile drivers, and dredges, are subject to some of the same corrosive influences as the steel piles described herein.

Corrosion of the steel occurs in four (4) zones. These are earth, water, tidal and splash zones. Fig. 1 illustrates these zones.

The most intense corrosion activity is usually found in the tidal zone where the corrosion rate may be as high as 0.030 in. per yr. (30 mils per yr).

A somewhat lower corrosion rate is found in the water or continuously submerged zone with rate varying from 5 mils per yr (mpy) to 20 mpy. These corrosion rates are on the basis of uniform corrosion and do not consider pitting corrosion rates which may be from 1.5 to 10 times as great as the uniform corrosion.

The splash zone rate of metal deterioration is largely dependent on whether there are contaminating agents in the water. For example, clean sea-water will promote its highest rate of corrosion in the splash zone, reaching values of 15 mpy. If this sea-water is contaminated with oils, sewage or other organics, the corrosion will be somewhat decreased in the splash zone due to protective film coatings on the steel surface, and may be even lower than the corrosion rate in the water or submerged zone.

The lowest corrosion rate will usually be found in the earth zone. However, highly reducing soil bottom (anaerobic) beneath highly oxidizing water (aerated) can encourage the kind of corrosion which results in rapid metal loss and intense pitting corrosion.

Some corrosion also takes place on the re-inforcing steel used in concrete above the water level. Sea-water salts are introduced into the concrete. The points of high salt concentration are the anodic sites of corrosion cells. Where the sea-water salts are uniformly distributed throughout the concrete, the points of highest moisture content become the anodic sites of corrosion cells.

If the corrosion rate of a particular structural member in a particular environment can be determined, and if it is possible for the design engineer to determine the amount of metal loss which a steel member can sustain, then an estimate of the life of the member can be made. Often, many assumptions must be made to determine what metal loss can be sustained.

In some cases, the engineer may overdesign the structure with the idea that the added metal thickness will add life to the structure. This can be an uneconomical solution, and may result in costs of at least 35% higher than

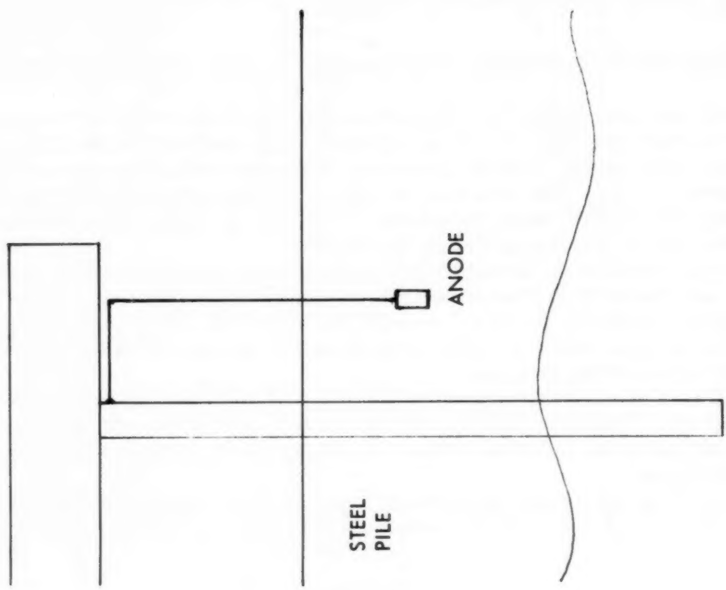

FIG. 2.—GALVANIC SYSTEM

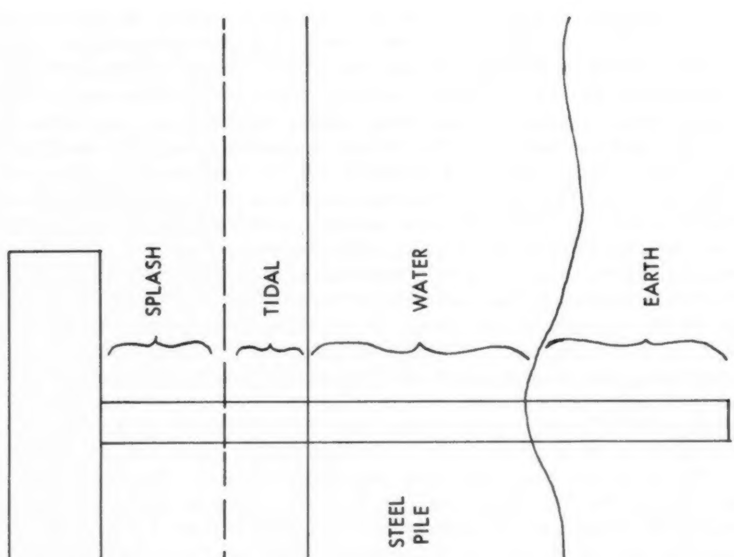

FIG. 1.—ZONES OF CORROSION

cathodic protection methods used for controlling corrosion with no way of extending the life, if this is contemplated.

CORROSION PREVENTION, COSTS MATERIALS OF CONSTRUCTION

Some of the methods for controlling corrosion involve a choice of material such as coatings, platings, sheathing, or cladding the steel member. Coatings may range from paints, enamels and gunite to the new coal-tar epoxy resins. Encasement of steel with concrete is also a common practice. Platings on steel may be applied using the molten metal and spraying on the surface. Aluminum or zinc may be applied in this manner.

Sheathing consists of forming a non-deteriorating material onto the steel surface and fastening in place. The material might be a reinforced polyester type of plastic sheeting or a long life metal such as Monel. All joints are made watertight. In some cases, a mastic may be forced into the space between the steel and the sheathing material.

Cladding refers to the metallurgical bond application of one metal to another by rolling on in the mill. In this way, stainless steels are rolled onto structural steel sheet and plate. Clad materials are usually too expensive to use except in special cases.

Up to now, we have been concerned with situations in which corrosion may be a problem, and some of the methods of allowing for it or decreasing it.

CATHODIC PROTECTION

Cathodic protection is the prevention of corrosion of metal in an electrolyte (soil or water) by the flow of direct current through the electrolyte to the metal surface. The current is introduced into the electrolyte by electrically conductive materials or by galvanic metals, such as magnesium or zinc.

The electrically conductive materials might be metal or graphite connected to the positive terminal of a direct current source. The steel to be protected is then connected to the negative side, and the steel is under protection. The current obtained by connecting galvanic metals such as magnesium or zinc to the steel results in the same electric current effect on the surface of the steel, and the steel is again under cathodic protection.

Examples of steel under cathodic protection are shown in Fig. 2. In this case, current, furnished by the corroding of the galvanic anode, flows through the water to the surface of the steel. At the steel surface, the steel is discouraged from corroding because of the reducing environment created by the electric current. The formation of alkaline compounds here also creates a corrosion inhibiting environment.

Fig. 3 shows steel being protected by impressed current anodes. The current is furnished by an exterior source of power rather than from a corroding process. The power is generally from metallic rectifiers. The prevention of corrosion occurs in the same way as with the galvanic anode, by current flowing to the steel surface. In addition, it is possible to have a film of hydrogen formed because of the availability of higher voltages. The hydrogen serves to isolate the steel from the water and, thus, prevent corrosion.

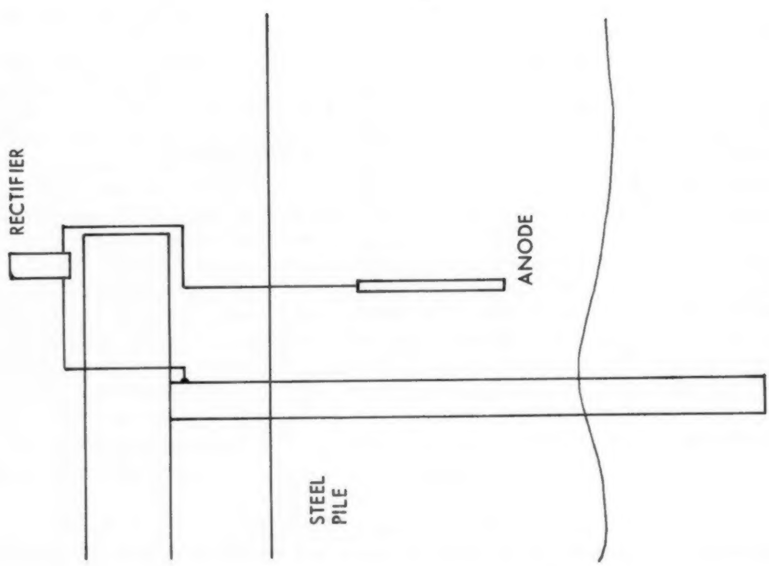

FIG. 3.—IMPRESSED CURRENT SYSTEM

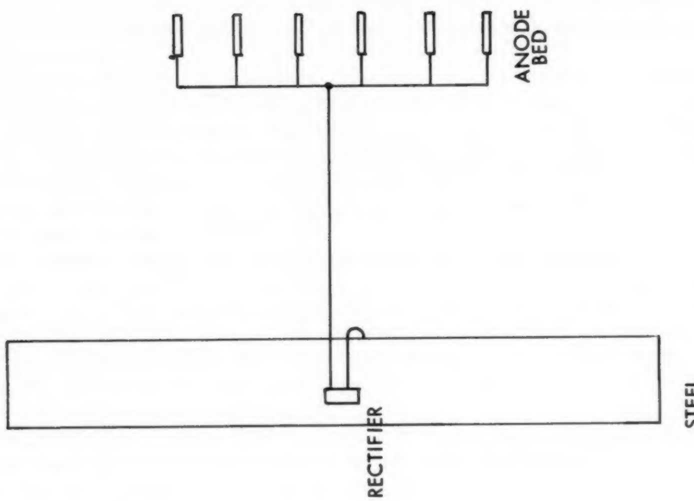

FIG. 4.—ANODE BED

Sometimes, several anodes are placed together to form a string of anodes.

Fig. 4 shows a typical installation of impressed current anodes protecting steel piles in a pier. These anodes might be either graphite or cast iron and up to 3 in. diameter by 60 in. long.

Cathodic protection anodes may be either suspended in the water or allowed to be on the bottom. If the anode must be close to the metal, as in the case of

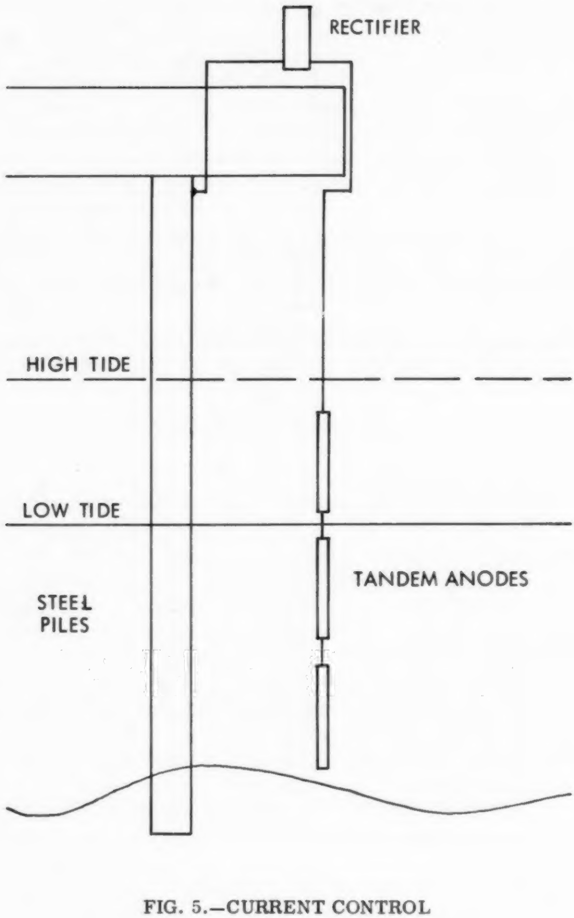

FIG. 5.—CURRENT CONTROL

the protection of numerous "H" piles in a pier, the anodes should be suspended in a string, sausage fashion, parallel to the steel surface. This will give the most uniform current density. If the anodes are to be placed near the foot of the pile and on the bottom, the current density on the steel will not be as uniform as when suspended.

When it is possible to place the anodes remotely from the steel surface, as in the case of a quay wall, the anode may be either suspended or laid on the bottom. Suspended anodes cost more for installation and may be troublesome with floating debris vessels or ice. Anodes on the bottom will not be affected by such conditions. On the other hand, electric power costs are usually higher for anodes placed on the bottom, due to the greater resistance of the earth compared to the water.

The many variables which affect the design of a cathodic protection system can usually be considered by gathering information during a survey. The survey consists of determining the extent of the structure to be protected, its method of construction, nearby structures, the type of corrosive environment and other data. On some occasions, however, the data show that some factors affecting the design of the cathodic protection system change from hour to hour or from season to season.

For example, when the steel area being protected changes from hour to hour, as in the case of tide changes, it may be necessary to have some type of control over the current output. These controls may be fancy or simple, depending on the situation. Often, it is possible to install the anodes in such a way that as the tide rises and brings more steel area into the cathode circuit, the same tide covers more anode length and increases the current output. This may be illustrated as shown in Fig. 5.

The advantage of this system, providing it has been properly designed, is that the steel is not overprotected at low tide with resulting coating loss, and loss of electric power. Also, at high tide, the steel is not underprotected with the resulting corrosion.

Another factor which may change from season to season might be the resistance of the water or earth. For example, a particular harbor, bay or river might ordinarily contain sea-water. But during rainy seasons or during a thaw, the water may be brackish or even fresh. Brackish water and fresh water are higher in resistance than sea-water. For a given anode voltage, less current will flow in brackish water than in sea-water. Thus, the steel will not receive proper protection current. This effect is worsened by the knowledge that steel in brackish waters requires higher current densities than in sea-water for the same level of protection.

With a changing water resistance, there is no simple mechanical design, of the protective system, to consider this change. However, it is possible to design the cathodic protection system to furnish protection under conditions of changing water resistance. One way of doing this is by the use of relays which vary the rectifier output to keep the protection current within a desired range. Some rectifiers are now available which keep a desired current output within certain ranges, although the load (anode) resistance varies.

The foregoing brings to your attention some of the technical aspects in regard to design of cathodic protection systems for marine structures.

CONSTRUCTION CONSIDERATION

Now some of the construction considerations with which the engineer may be concerned with in regard to cathodic protection will be examined.

The most important construction aspect is metallic continuity of the structure. This means that all portions of the structure which are to be in

water or earth must be metallically connected. This connection, in many cases, arises naturally due to welding together of members, bolting and the use of reinforcing steel.

The "H" piles which are connected by steel members through welding are considered metallically connected.

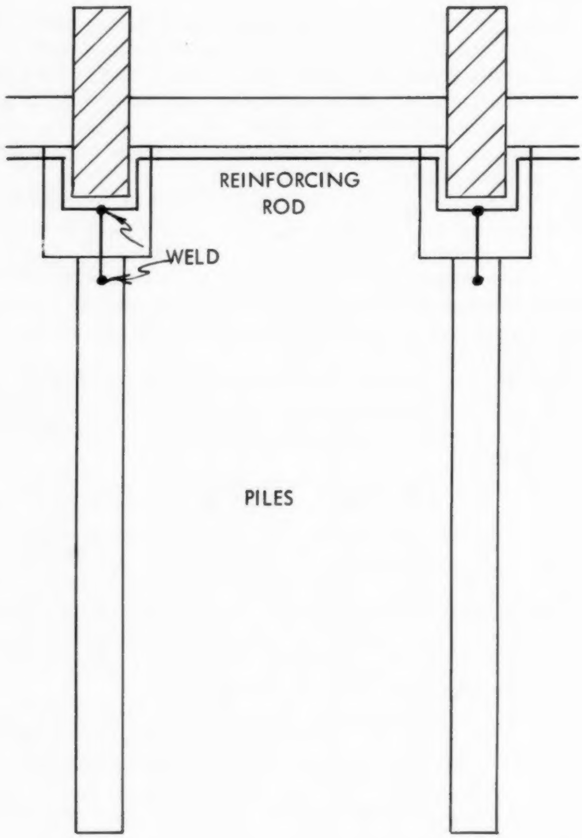

FIG. 6.—PILE BONDING

The "H" piles which are connected by reinforcing steel through welding are considered metallically connected. Fig. 6 illustrates how additional bonding could be done if a structure is already built.

Sheet piles connected by steel members or reinforcing steel through welding are considered metallically connected. Sheet piles connected mechanically at the bulb and groove may be connected or not, depending on the condition of

the mating surfaces. K. A. Spencer suggests[2] insuring electrical continuity on sheet piling by 6 in. of fillet welding between all piles of a sheet-piled structure.

If steel members are metallically connected only through reinforcing mesh, this may not be adequate. The size of metallic bonding connections are determined on the basis of conditions of the structure and its environment.

Bridges which are supported on caissons or reinforced concrete footings should have the metal bonded together and connected to the bridge structure.

Underwater utilities in the vicinity of the marine structure must be considered. Cathodic protection current collected by the utilities must be drained through a metallic connection to avoid stray current corrosion. The manner of making the connection and value of the connection resistance differs from situation to situation. Usually, these adjustments are made during the start-up of the cathodic protection system.

For structures capped in concrete, it may be desirable to extend the concrete to mean low water as a means for preventing corrosion in the tidal zone. This feature must be weighed against anticipated corrosion damage in this zone. Structures near the open sea are likely prospects for this type of construction, whereas marine structures more inland in harbors and rivers are less likely to benefit from concrete encasement. It is important that, if concrete encasement is used, the concrete not extend below mean low water. If the concrete extends below mean low water, a corrosion cell will be set up between the anodic wetted steel and the cathodic concrete encased steel.

Test leads for future testing of the effectiveness of the cathodic protection system should be installed at the time the marine structure is constructed. Insulated conductors should be brazed to the structures at locations between current drain points or between anodes. The purpose of the test lead is to provide an easy way of making metallic contact with the steel. The lead should terminate within a junction box.

In addition to test leads, it is presently the practice to provide reference electrodes, such as zinc, to use for testing the potential of the steel. This is accomplished by submerging the zinc electrodes at various points near the structure. A cable from this electrode and the test lead from the steel structure are sufficient for measuring the potential of the steel, using a portable voltmeter. The potential gives an indication of the degree of protection being provided. In some cases, a permanently installed voltmeter indicates the structure potential by the mere push of a button. This is called a potential indicator.

When rectifiers are used for the source of power, the proper operation can be signified by a glowing red light. When the protection current drops to a low level, a green light glows to indicate a malfunctioning. In some cases, horns or bells are used to give the alarm for failure of the corrosion control system. Such controls are finding favor in those cases where a few individuals must be able to instantly recognize the failure of various types of electrical, mechanical, or chemical equipment.

Piers which serve to fuel or defuel ships present a special problem in regard to cathodic protection systems. The potential gradient existing in water where anodes are installed causes current to flow in the water. Thus, a voltage

[2] "Cathodic Protection in Relation to Engineering Design," by K. A. Spencer, Chemistry and Industry, 1953, pp. 2-10.

of 6 DC impressed between an anode and the cathode on protected structure causes current to flow from the anode through the water, and onto the protected steel surface. The current follows many paths through the water. If plotted, these paths resemble the magnetic field surrounding the poles of a magnet. When a ship approaches a pier, its hull intersects these potential gradient lines. As the hull is a better conductor of electricity than the water, the current flows in the hull along its intended path until such point as it is necessary to re-enter the water and return to the pier. In flowing onto the hull and then flowing off, a voltage is set up between the ship and the pier. It is this voltage which is a hazard in the vicinity of explosive atmospheres.

One solution to the problem is to de-energize the cathodic protection system. This will remove the potential gradient which caused the problem. This is a practical solution in those cases where the pier is active a small percentage of the time. However, no protection against corrosion is received with the system turned off. Turning the system off would be impractical in those instances in which the pier is active with fuel operations a substantial part of the time.

In the latter case, provision should be made for bonding the ship to the pier structure. To avoid the possibility of a spark at the instant the bond is made, the connection is made inside an explosion-proof switch (Class I, Group D). One way such a bond is accomplished would be as follows:

1. As the ship approaches the pier, the bonding cable is attached to the ship structure.
2. The explosion-proof switch is closed.
3. Fueling operations are begun.

An improvement on the previous system would be meters to indicate the status of the bond. This would be a simple voltmeter to show that a potential existed between the ship and pier before bonding, and a simple ammeter to show that current was flowing from the ship to the pier after bonding. As additional metallic connections are made between the ship and pier, the bonding cable would carry only part of the current. As long as the current flowed, partial cathodic protection would be offered to the ship. In the case of well-coated hulls, the ship would be given more protection than if the hull were poorly coated. As to the ship's affect on the pier cathodic protection system, it is well to emphasize that some loss of protection may occur. However, the amount of loss will generally be low. For example, piers at Harbor Island, Texas lost approximately 0.05 v and 30% of the protective current due to the docking of a 660 ft tanker.[3]

The bonding cable design is worked out on a basis of the potentials encountered at the pier, the quantity of current to be drained and the ignition energy of the flammable mixture.[3]

INSTALLING ANODES AND PROTECTING THEM

Galvanic anodes are generally of shorter life than impressed current anodes. Two hundred fifty pound zinc anodes have a life of approximately 10 yr in sea-water, and 50-lb magnesium anodes will last approximately 1 yr.

3 "Electrical Significance of Cathodic Protection on Hazardous Area Steel Docks," by T. A. Mullett and J. W. Johnstone, Jr., Corrosion, May, 1960, p. 85.

These anodes are rugged and easily installed. A coated steel rod or wire rope serves the purpose of suspension and electrical connection. If the anodes are to be placed on the bottom, only an insulated electrical connection is required. Any small nicks in the insulation will cause no harm. This is because the exposed copper will be cathodically protected. The anodes are not particularly brittle, but care should be used in handling as with any type of metal casting.

Placement of the anodes, whether suspended or on the bottom, should be as shown on the engineering plans. In those cases in the field where structural members, such as angles, channels, or tie rods would be touching the anode, then the anode should be moved away so as to avoid current wastage to this localized area. The idea is to place the anode as centrally as possible for the area of steel which it is to protect.

Galvanic anodes are furnished with steel cores or an insulalted copper conductor. The suspension rod (or rope) may be brazed to the steel core. This rod should then be covered with plastic tubing before installing the anode, so as to prevent current wastage.

Impressed current anodes are longer lived than galvanic anodes, providing the system has been properly designed. They might be either graphite or cast iron, although low carbon steel, stainless steel, lead, tantalum and platinum anodes have also been used successfully. Graphite or cast iron anodes are brittle and cannot stand as rough a treatment as zinc or magnesium. These impressed current anodes are furnished with insulated leads. Utmost care must be used in handling the lead to avoid nicks, cuts, or scratches in the insulation. Unlike galvanic anodes, the impressed current anode lead wire is not cathodically protected. Any penetration of the insulation will lead to rapid loss of the copper when operating with consequent loss of continuity to the anode. Likewise, where several anodes are spliced to a single header cable, the splice must be waterproof for the life of the installation. This point cannot be overstressed. The loss of one splice might result in the loss of an entire string of anodes. This is one point where quality of workmanship pays large dividends in the life of the installation.

Generally, the impressed current anodes are suspended by their own lead wire. Sometimes, several anodes are on the same lead wire, sausage fashion. However, to prevent strain on the lead wire, the anode may be suspended by some other means using non-metallic materials such as manila rope or certain plastic ropes. To allow this method of suspension, some manufacturers make their anodes with a hole near the end of the anode. If the anodes are to be placed on the bottom, the electrical lead is usually strong enough to allow installation.

The anodes used in impressed current systems are at higher voltages, and are generally capable of higher current outputs so, for the same situation, they will protect a greater amount of steel surface. Because of the higher voltage, accidental touching between the anode and the protected steel is more critical, and more care should be exercised in placement of the anode to avoid shorting effects.

Some marine bottoms consist of hard earth which will support anodes. If the bottom is soft, the more dense anodes such as cast iron or zinc will gradually sink. This causes no harm, and helps to protect the anode. However, any scrap metal or sharp rocks on the bottom may damage or sever the anode cable as the anode sinks. Slipping a plastic tube over the anode lead may prevent such damage.

Scrap metal, such as sheet, plate or shapes should not be dumped in the vicinity of marine structures to be cathodically protected. Such metal will divert the protective current and prevent proper protection of the structure.

Cables for the anodes should be installed to avoid later damage from buried structures, utilities or material. The installation should be away from areas to be dredged or where future construction is to take place. As an added precaution, a cable may be protected by pulling it into a plastic conduit. This increases the life where mechanical damage is likely to occur.

Cables from rectifier units should be in conduit where they enter the water. For instance, on "H" pile structures, a positive lead from a rectifier unit to anodes can be protected by pulling into metal conduit and welding the conduit to the web of the "H" pile.

In an ideal situation, the corrosion engineer enters the picture during the planning stages. At this point, he can determine what general problems are to be encountered. He may be in a position to evaluate various ways of insuring the life of the structure.

If, during this planning stage, it developes that some means should be taken to combat corrosion, the engineer will be in a position to mesh the corrosion engineering with the engineering of the project.

If a structure in a corrosive environment is already in place, it is still possible to determine means of extending the life. The most economical means should be determined, considering the conditions which would influence the decision. These conditions include initial cost, maintenance cost, replacement cost, and desired life.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

COLUMBIA RIVER SHIP CHANNEL IMPROVEMENT AND MAINTENANCE

By Robert E. Hickson,[1] F. ASCE

SYNOPSIS

A simple practical solution for design of channel improvement and control works on the Columbia River, and the beneficial effects of such works are described. Also considered are alternate authorized methods and plans for channel maintenance at the mouth of the river with comparative costs over a hypothetical life of 50 yr.

INTRODUCTION

The improvement and maintenance of open river channels for navigation has engaged the attention of engineers since early historic times. It is an interesting field of engineering as each project is in many respects different from the other, and the end product, a satisfactory channel, is under water where it cannot be seen, in contrast with other engineering works.

ORIGINAL CONDITIONS AND EARLY WORK

Originally the channel of the lower 110 miles of the Columbia River had controlling depths of 12 ft to 15 ft at low water on several bars. Some dredging was done for relief between 1866 and 1876. The first project for a depth of 20 ft was authorized in 1878. The project was later increased to 25 ft, and in 1912

Note.—Discussion open until January 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 2, August, 1961.

[1] Cons. Engr., Formerly Chf. of Engrg. Div., Portland, Oreg. Dist., and Chf. of Const. and Operations, North Pacific Div. Corps of Engrs., U.S.A.

to a depth of 30 ft and width of 300 ft. In 1930 the present project, for 35 ft by 500 ft, was authorized.

The first work by the Federal government to effect a permanent improvement of the channel was at St. Helens Bar where in 1878, a longitudinal dike was constructed from the right bank and extended at an angle downstream for a length approximately 6,200 ft. Other works were constructed with Port of Portland funds at Martin Island, Walker Island, and Snag Island (the latter in the head of the estuary).

These early Columbia River dikes were at widely separated locations, and not under a comprehensive plan. They were accordingly only partially successful. Some of them have, however, been incorporated in and constitute a part of the present system of control works. At the mouth of the river the entrance was divided into several channels with overlapping sand spits. The channels shifted rapidly from year to year over a wide arc of approximately 6 miles, and depths in the crooked channels ranged from approximately 20 ft to 24 ft at low water.

GENERAL DATA FOR COLUMBIA RIVER

Following are some of the principal data for Columbia River:

Source, Lake Columbia, Canada:	
Area of drainage basin	259,000 sq miles
Length of Columbia	1,200 miles
Total fall	2,640 ft $\pm$
Average annual runoff	178,000,000 $\pm$ acre ft
Average annual freshwater discharge	243,000 cfs
Maximum freshet flow	1,260,000 cfs
Minimum freshwater flow	37,000 cfs
Flood stages maximum at Portland (before storage)	33 ft
Flood stages average at Portland	20 ft +
Tides at Mouth, Datum MLLW	0.0
Maximum	10.5
Average	7.5
Minimum	-2.5
Tides Range at Portland, Low Stage	2.0 +
Tidal reach	150 $\pm$ miles

The foregoing data and those given in subsequent paragraphs are presented to show the size and some of the characteristics of the Columbia River. Fig. 1 shows the lower 140 miles of the river.

LOW WATER SLOPES

Total fall of the river from the Canadian border to the sea is 1,300 ft, but the fall at low water datum from Portland to Astoria is a little more than 5 ft, an over all slope of approximately .05 ft per mile for this reach. This slope does not exist, however, except for short intervals and relatively short reaches at any time, as the tidal effect is continually changing the slope. The low water plane and theoretical slope is therefore really the envelope of the low tides at various points for the low stage of the river.

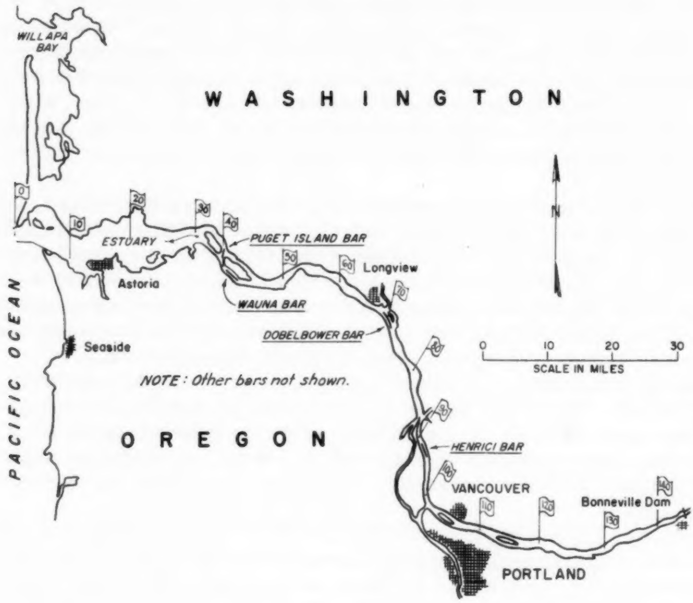

FIG. 1.—VICINITY COLUMBIA RIVER BONNEVILLE TO THE SEA

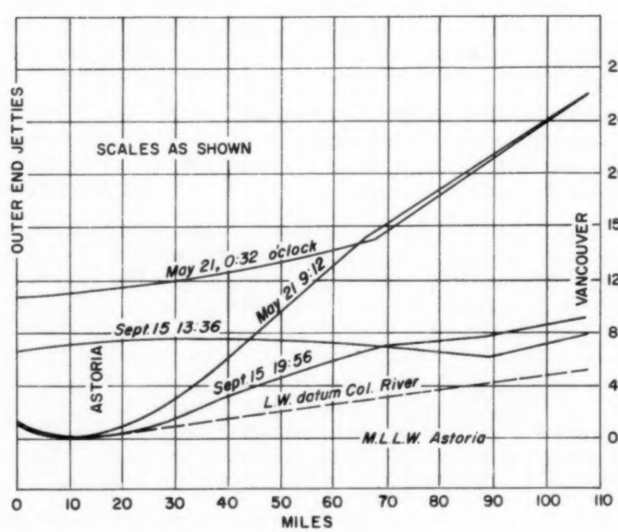

FIG. 2.—WATER SURFACE PROFILES - 1932

There are reversals of slope which produce up river currents for 75 miles to 80 miles above the mouth at low river stages of flow. The water surface slope during ebb phase of the tide was found from simultaneous readings, May 21, 1932 at high river stage, to be approximately 0.35 ft per mile, at points 40 miles to 60 miles above Astoria. At low river stage in the same reach the ebb slope was approximately 0.14 ft per mile. There is a node for tides at approximately mile 67. Fig. 2 shows profiles of lower river as affected by stage and tides.

Velocities of River Flow Above the Estuary.—In any given stream, velocity is the principal determining factor in maintenance of channel depths. This is, however, dependent on channel section, slope, volume of flow, roughness of the bed, tides, etc. But the parameters slope and volume, are dependent on freshet flows and tidal effect. The freshwater flow varies from day to day and year to year, and the tidal effect is also quite variable through the ranges of neap and spring tides and on account of diurnal inequality when present.

Observations made on the lower Columbia, above the estuary show velocities during freshet stages ranging up to approximately 7 fps near Vancouver. At low stages, the velocity in the channel at that point is on the order of 3 fps. Tidal action at low river stages causes a reversal of current up to approximately 80 miles above the mouth, above which point the current is always down-stream.

Tides and Tidal Velocities at Mouth.—At the mouth of the Columbia the mean range of tide is approximately 7.5 ft with extreme highs of approximately 10.5 ft and extreme lows of -2.5 ft. There is a marked diurnal inequality under spring tide conditions. During spring tides there may be a total drop from higher high to lower low water, of 13 ft more or less in a period of 6 hr of ebb flow. There is never as great a range for the flood phase, and there is accordingly a great preponderance of the ebb over the flood tide both in maximum velocity and volume of flow. The ebb is also increased by the volume of river water impounded during the flood phase as well as the head flow from upstream during the ebb.

The following figures were obtained from the current survey of 1932:

Highest average ebb velocity (at high stage)	7.2 fps
Highest peak discharge (at high stage)	3,065,000 cfs
Highest average flood tide velocity (at low stage)	4.96 fps
Highest peak inflow (at low stage)	2,048,000 cfs

At high river stage and large ebb phase range, observations recorded a maximum velocity of 12.5 fps at 1/10 depth. The maximum flood tide velocity recorded was 7.0 fps at 3/10 depth at low river stage. The figures for bottom velocities listed in Table 1 are from the current survey of 1932. More complete data and an analysis of tidal currents are presented elsewhere.[2]

Density and Littoral Currents.—As usual, at the mouths of rivers entering the ocean, a density current at and near the bottom, which flows inland is established. This is in the form of a salt water "wedge" under the fresh water moving seaward on the surface. At the mouth of the Columba, this is evident at and immediately after the turn of the tide from ebb to flood for a period of approximately $1\frac{1}{2}$-hr. With the $7\frac{1}{2}$-ft tidal range at the mouth of the Columbia

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[2] Discussion by R. E. Hickson of "Interim Consideration of the Columbia River Entrance," by J. B. Lockett, Proceedings, ASCE, Vol. 85, No. HY 8, August, 1959, p. 95.

River, however, the fresh water out-flow at ordinary stages is entirely stopped, the flood flow is largely of salt water from surface to bottom, and the salt water wedge phenomenon as such is gradually forced upstream several miles. On the succeeding ebb, the salt water is almost entirely washed out to sea again, and currents are seaward throughout the full depth. With the turn of the tide the cycle is repeated. Observations in 1936 and during the survey of 1959 show that salt water intrusion on the bottom extends approximately 11 miles above Astoria.

Observations at three different river stages made during the comprehensive current survey in 1932, showed the predominance of the ebb over flood bottom velocities at higher river stages, with an approximate balance at intermediate and low stages.[3]

Another comprehensive current survey was made by the Corps of Engineers, of the Portland, Oregon District in 1959-60, with more modern equipment to determine velocities, directional flow and other data. A review of this survey report indicates that the findings as to currents are in substantial agreement with those of the 1932 current survey.

Ocean littoral currents are the cause of major shifting of material, but they are not always in the same direction, probably changing direction with the

TABLE 1.—BOTTOM VELOCITIES (AT 9/10 DEPTH)

River Stage	5% of Time		40% of Time	
(1)	EBB, in fps (2)	Flood, in fps (3)	EBB, in fps (4)	Flood, in fps (5)
Low	2.8	2.86	0.9	1.1
Intermediate	2.7	2.72	0.82	0.7
High	3.6	2.52	1.42	0.20

seasons or prevailing winds. While the shifting material may be a major or controlling factor at smaller inlets, it is probably of secondary importance in the case of a large inlet such as the Columbia River, where the ebb tidal flow combined with river flow creates velocities high enough to remove any material transported along shore to the inlet by littoral currents.

The velocity of littoral currents is generally relatively low as compared with ebb velocities through the inlet and may require the stirring effect of wave action to put bottom materials in motion. The ebb tide bottom velocities in the channel of the Columbia River are of a much higher order. When it is realized that the tractive force which moves materials on the bottom varies with probably the square of the velocity, it can be understood how at inlets with large flow, the ebb tidal currents will prevail over movement of material into the channel by the lower velocity shorewise littoral currents.

At small inlets with small tidal flow and low velocities, littoral drift becomes a major consideration, and may require bypassing to the down-drift beach across the inlet channel as is done at some inlets by operation of submerged pipeline dredge equipment.

[3] Ibid., Fig. 1.

Suspended and Bed Load Transport.—A current and sediment survey was made in 1922 in the Columbia River below the mouth of Willamette River, to determine the amount of material carried in suspension during 13-ft and 20-ft freshet stages. Samples were taken with a specially designed trap at several points in the verticals and several verticals in the section. These showed an average of approximately 130 ppm of flowing water, before dam construction.

On the basis of these observations and the flow record of the river during a period of 78 yr, using the flow during the five higher months each year, it is estimated that under original conditions there was an annual discharge of approximately 15,500,000 cu yd of suspended material. With the construction of dams on the upper river, it is probable that a large part of the suspended material is now being trapped in the reservoirs, although surveys in the Bonneville pool for several years did not show conclusively significant deposits. Probably the larger deeper pools with lower velocities trap more of this sediment. If it is estimated (not based on any survey data) that approximately 1/2 of the suspended material, 7,500,000 cu yd, are retained annually, there would remain approximately 8,000,000 cu yd which passes out to sea annually as suspended material.

The measurement of bed load movement is more difficult. The annual movement below the dams in the years since construction of control works above the estuary has been at a greater rate than was the case when the river was in its natural condition, as scoured material has been added to the normal movement. This increased rate will continue, however, only until the improved sections again reach a state of equilibrium (which they are now approaching). An approximate estimate of the volume scoured from the lower river improved channel section based on the surverys at Henrici and Dobelbower bars and more detailed estimates for the channel between Vancouver and Bonneville, also under dike control, indicates a total movement of approximately 140,000,000 cu yd in about 40 yr, from areas outside the dredge cut channels. This indicates a rate of approximately 3,500,000 cu yd per annum transported by river bottom currents as bed load. Adding this bed load of scoured material to the suspended load accordingly makes an estimated total of approximately 11,500,000 cu yd of material passing to sea annually.

In 1922 the writer estimated from surveys that the net deposits on the Columbia Bar outside the jetties was approximately 4,000,000 cu yd per annum.[4] It therefore appears that of the approximately 11,500,000 discharged from the river annually, about 7,500,000 are dispersed in movement by littoral currents, wave, and wind action, and in the deep water farther at sea. The outer bar at the 50-ft depth has advanced approximately 2 miles to sea since 1885 (Fig. 9(A) presented subsequently).

It is not possible to definitely fix the final resting place of the river transport, except it is known that, no matter what its exact volume, it has gone into the sea to be dealt with by the various forces of nature. In the previous centuries, the total amount of material discharged may be built up to almost any figure. The extensive deposits of sand in successive sand dune ridges along the coast line between the Columbia and Seaside and in the Long Beach peninsula north to Willapa Harbor, contain tremendous quantities of sand and silt which apparently came from the Columbia River, as claimed by geologists. Petrographic analysis of the sands also indicates this source.

4 "Changes at the Mouth of the Columbia River, 1903-1921," by R. E. Hickson, Military Engineer, May, June, 1922, pp. 211-214.

The figures given for river transport are necessarily rough estimates, but are considered to be for the right order. There are new influences now at work, such as reservoirs on the upper river, and dike control works, all of which tend to change any earlier estimates. Reservoirs will decrease the volume for many years while the effect of control works is to increase the bed load movement through scour in the lower reaches under control until equilibrium has been reached. Considering all these influences it appears that in future years the gross movement should be less. However, the specific volume of solids being transported by the River is not a controlling factor in that it may affect the maintenance of the channel where it is under control. Where it is fully under control as to section and velocity necessary to a stable section, all material being transported will have to pass on to an area or section where equilibrium has not been established (see the section entitled "Engineering Aspects and Problems" given subsequently).

Disposition of Transported Materials.—To determine whether any appreciable part of the material moving down river is being deposited in the estuary, the Corps of Engineers of the Portland, Oreg. district made a detailed comparison of two hydrographic surveys of a large part of the estuary. The earlier survey was made in 1868 by use of a lead line and shows soundings to the nearest 1/4 fathom. The other was made in 1958, on which a fathometer was used. Both surveys were made by the United States Coast and Geodetic Survey (USC and GS).

This comparison indicates shoaling amounting to 77,000,000 cu yd in the 90 yr period. Due to the inherent difference in the methods of sounding, however, the two surveys are not strictly comparable. Lead line soundings, for several reasons generally recognized, indicate greater depths than actually exist, while the fathometer on the other hand picks up the high points on the bottom, thus indicating less than the average depth.

However, not considering these differences, which might largely account for the indicated deposit, it is found that a deposit of 77,000,000 cu yd in 90 yr would account for only a small part of the total river transport of solids in that time. On the basis of the annual river discharge of 178,000,000 acre-ft, the deposit indicated by the two surveys is less than 3% of the estimated total movement of solids. Under different assumptions this percentage might be somewhat increased but it would still be small. The foregoing shows that solids being transported by the river all eventually go to sea, as of course they must.

As to the possible effects of up-river improvement works on conditions in the estuary, Table 4 shows that channel dredging in the estuary has actually decreased in the past 10 yr.

Bed Materials.—Table 2 shows screen grading of sands found in channel dredging in 1928 at Morgan and St. Helens bars, 7 miles and 17 miles, respectively, below Vancouver.

Many samples of bed materials were taken at various places in the lower river and at the mouth by the Corps of Engineers during the survey of 1959. The grading varies depending on locations of the samples as to the channel and configuration of the bottom.[5]

The results of screening of 200 samples of bed materials at the mouth of the Columbia are also shown in the current survey report of 1932. The mate-

5 Interim Report, Current Measurement Program, Corps of Engrs., Portland Oreg. Dist., Vol. 4, Plates No. 304 to 309, 1959.

rial is generally finer than up-river sand with a large part in the 50 to 80 mesh screen sizes.

Banks of the lower river are of fairly durable clay deposits laid down by the river and tide land deposits. For considerable distances one bank or the other is of rock. All in all the banks may be considered as fairly stable although considerable erosion takes place at a slow rate. On the upper reaches of the river the bed and banks are mostly gravel and rock.

Control Works.—In the river channel between Vancouver and the head of the estuary, there have been a total of 150 spur dikes or groins constructed principally in 1916 to 1930 under the 30 ft project. Some estenstions of the original and additional dikes were added for control under the 35 ft project.

The dikes, in general, are of the permeable type, built of piling and stone. The piles are driven in two rows in a staggered arrangement with a spacing

TABLE 2.—SCREEN ANALYSIS OF SANDS

Screen Meshes per in.	Morgan Bar		St. Helens Bar	
	%	Cumulative	%	Cumulative
(1)	(2)	(3)	(4)	(5)
4[b]	0.0	0.0	9.1	9.1
8[b]	0.3	0.3	7.3	16.4
10[b]	0.3	0.6	3.6	20.0
20[b]	6.8	7.4	20.8	40.8
30[b]	8.7	16.1	19.7	60.5
40[b]	21.0	37.1	26.4	86.9
50[b]	28.1	65.2	8.7	95.6
80[b]	31.5	96.7	4.2	99.8
100[b]	1.0	97.7	0.1	99.9
100[c]	2.3	100.0	0.1	100.0
	Voids	38.0%	40.0%	
	Specific Gravity	2.51	2.63	

[a] Mil Engr. May-June 1930.
[b] Retained on.
[c] Passing.

generally of $3\frac{1}{2}$ft in each row. The two rows are fastened together with a longitudinal separating timber and a waler near low water level. Stone is placed on the bottom along the piles to stiffen the structure and prevent scour. Riprap protection is also placed at each end.

Elevation of pile cut-off varies from 10 ft at upper reaches to a minimum of 6 ft in the lower river. Ten ft is approximately $\frac{1}{2}$ the flood stage in upper reaches. In a few cases where pile dikes are used for bank protection, the cutoff is set at almost 1 ft below bank level, and spacing of piles is widened to 5 ft in each row to avoid eddy action. Dikes are also spaced closer together.

The permeable type of construction, and limited height of control dikes are used in order to avoid initial over contraction, which would set up excessive currents. This would cause large volumes of bed material to be set in motion, in the early stages of improvement, before the section has been sufficiently enlarged to carry the flood stage flow without undue disturbance.

The spacing of dikes will vary considerably, depending on several factors, and whether primarily for contraction or to assist in bank protection. On the Columbia River spacing is about 800 ft for bank protection, and 2000 ft more or less in other situations. Curves on the lower Columbia are of long radius, and dikes can be spaced farther apart than would be the case if the channels had sharper curves. Spacing should be such that the scouring effect from each dike will carry to the next dike downstream or into a naturally controlled deep section. There is no well-established rule for the spacing. It will vary with the velocity and hydraulic mean depth, and the nature of bed material. High velocity and greater depth will allow greater spacing, but heavy or coarse bed material will call for closer spacing.

On short bars, such as Puget Island, only one dike on each side of the channel has proved to be sufficient.

The spur dike or groin type of layout has been found more effective than the longitudinal type. This is because of the agitating effect of the spurs in putting heavy bed material in motion farther into the channel than the smoother longitudinal layout. The longitudinal plan allows high flows to pass down stream behind the dike. For the same amount of contraction the spur dikes will also generally cost less to build and maintain.

In excessively wide reaches, it has been found advantageous to build up middle grounds or islands, retained by cross dikes, thus placing the ends of dikes closer to the channel and insuring the agitating effect. Such layouts have been used at St. Helens, Eureka, and Wauna Bars.

The 30-ft Project.—The 30-ft project authorized by the R & H act of July 25, 1912 provided for a channel 30 ft deep and 300 ft wide, to be secured and maintained by dredging and the construction of pile dike control works at an estimated cost of $3,770,000. This estimate included the construction of two 24-in pipeline dredges with attendant plant to cost $520,000.

In the project document, however, under which the work was authorized, it was stipulated that "permanent works should be used only to a limited extent and after careful study and observations of each locality and generally after dredging had been found ineffective."

After 3 yr or 4 yr of maintenance dredging, it was found that little progress was being made toward a permanent or reliable channel. Each annual freshet shoaled the dredged cuts to nearly the same depth as before and difficulty was experienced in restoring depths in the short time before the river reached a low stage in the fall; deep draft shipping was hampered.

The situation was carefully studied at each locality and construction of the planned dike control works was then started. The first of such works was built in 1916, and additional works were added as funds became available until the original system of contraction dikes was practically complete by 1928. A total of approximately 100 contraction dikes were constructed in the river at some sixteen bars; Puget Island bar to Vancouver, in that period.

The 35-ft by 500-ft Project.—An enlarged project calling for a channel 35 ft by 500 ft was authorized in July 1930 at an estimated additional cost of $1,366,300 (not including Willamette River work). This project contemplated the extension of many of the existing contraction control dikes and the construction of additional works where needed to provide for maintenance of the increased channel dimensions. The total number of contraction dikes is 150.

This project is now (1961) in a maintenance status and is well serving the increased ocean borne traffic.

Table 3 shows the growth of ocean steamer traffic since 1920.

THE ENGINEERING ASPECTS AND PROBLEMS

Most rivers, if undisturbed and having fairly stable banks, will establish over the centuries a regimen which is practically in a state of equilibrium so far as general channel conditions, depths, and cross-sectional areas are concerned. There will be shifting within the banks, and banks may erode so that in time the position of the river bed may change. But its capacity for transport of suspended and bed materials will remain relatively unchanged.

Under these natural conditions all material being brought down from the watershed passes on through to some area where equilibrium may not yet have been established. Generally this is in the ocean or large sea.

TABLE 3.—TRENDS IN OCEAN VESSEL TRAFFIC ON COLUMBIA RIVER[a]

Drafts, in feet	Year				
	1920	1930	1940	1950	1959
	Ocean Vessel Traffic, in short tons				
	2,900,000	6,700,000	6,600,000	9,062,000	11,400,000
	Number of Vessel Trips				
30' to 34'	6[b]	4[b]	7[b]	398	517
28' to 29'	11	87	89	422	361
19' to 27'		2,261	2,404	2,688	3,029

[a] From reports of U. S. Engineer Corps. The large increase in the number of deep draft vessels and ocean tonnage, since 1940, reflects the great capacity for deep ships under the conditions of the improved river channel below Portland, Oreg. and Vancouver, Wash.

[b] The deeper vessels in these years had to depend to some extent on tides and river stages.

However, even though the river bed may be in a state of equilibrium and will maintain its depths indefinitely, these existing depths may not and generally are not sufficient for the needs of navigation of a river under improvement for ship navigation. Here is where the river engineer is called on to find the best solution.

If dredging is done to secure the depths required in the naturally shallow reaches, and the spoil is disposed of outside the section, ther area available for flow is increased, and the mean velocity reduced below the critical, necessary for movement of bed materials. The reduced velocity in the enlarged section allows new materials from upstream to be deposited and the section shoals toward restoration of the original stable section.

Solutions.—Of the two methods of maintaining channel depths, (1) by dredging and (2) by section control works, only the latter is based on engineering principles.

Dredging is a simple and direct method of removing a shoal, but as previously indicated, it is only a palliative. It does not attack the cause and has to be repeated year after year to practically the same extent. Deep dredging in naturally shallow waterways may simply be creating a catch basin in which new material from upstream is deposited. There are certain situations, however, in wide estuaries or bays in which velocities are low, and bed materials are very fine and easily moved about by wave action and passing vessels, where total removal from the section by dredging may be the only or most economical solution.

In river channels where bed materials are coarse, however, repeated deposits of dredge spoil in areas near or along the shore may result in semipermanent reduction of the channel section, and improvement of depths.

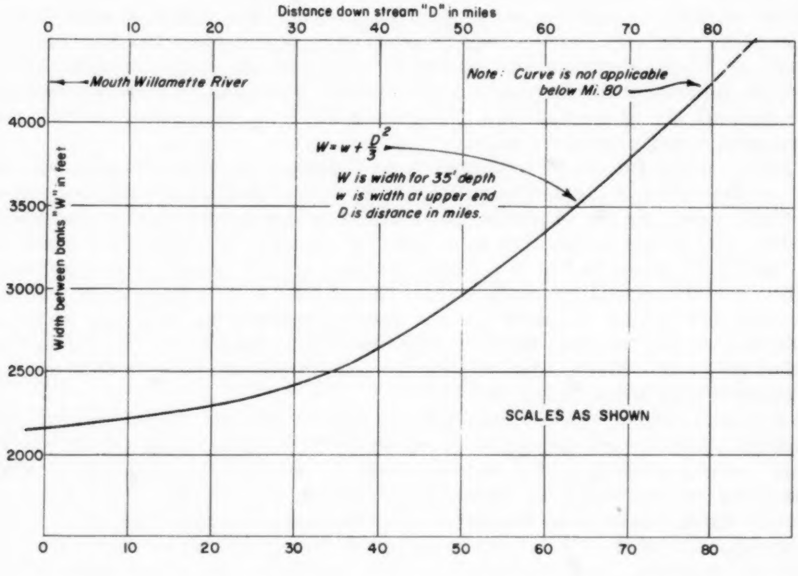

FIG. 3.—COLUMBIA RIVER - WIDTH FOR 35 FT CHANNEL

On the lower Columbia River, except in the estuary, the second method, by control works combined with dredging, as may be necessary, has been found most satisfactory. Under this plan the cross-sectional area is reduced to that necessary to produce ebb velocities high enough to prevent deposits in the improved channel. It is based on the fundamental relation, $V = Q/A$.

A more elaborate or theoretical analysis to determine the section necessary for maintenance, might be attempted by considering the effects of all the forces and combinations of forces which are at work, but it is believed it will be agreed by engineers familiar with river improvement problems that even an approximate answer by computations involving so many variables and unknowns is impracticable.

Hydraulic models are often used to aid in solution of channel improvement and maintenance problems. Here again many of the problems and difficulties are also present, especially in tidal waters. In movable bed models the problem is very complex. For fixed structures such as dams, spillways, stilling basins, locks, powerhouses, etc., models are a most valuable tool for the designing engineer and should be extensively used on such projects.

A Practical Approach.— As a practical solution, it is necessary only to accept the section produced by natural flow over a long period as being correct for the depth found therein. Nature is the perfect integrator of the effects of all the contending forces over a long period of time and the engineer can with confidence build or control the section to the dimensions so indicated.

To put this solution in practical operation, the writer used the following simple procedure. Hydrographic charts of the Columbia River made before any extensive improvements or changes had been made, were used to find as many sections as possible where the river, flowing in a single channel, had a natural section meeting the requirements as to depth. The width between the banks at these sections was scaled from the charts and the widths plotted against the distance downstream. A smooth curved line was then passed through the general lay of the plottings. The proper width at any point or section was then read directly from the curve.

Such a curve for the 35 ft channel of the Columbia River is shown in Fig. 3. Where the river is divided by an island, the total determined width is apportioned between the two channels, and the minor channel controlled at the upper end so that it will not enlarge as a result of constriction of the main channel.

The curve shown in Fig. 3 reflects the increasing flow and correspondingly wider section necessary down stream due to tidal effect. Constriction of the channel and filling of parts of the section reduces the tidal flow in some measure so that the final width required is slightly less at any point than that indicated by the curve. The empirical equation shown in Fig. 3 is of course applicable only to the Columbia.

A similar curve can be drawn for any similar stream if sufficient natural control sections are available from which to secure the data as to width. This method of determining channel width is not directly applicable to wide estuaries or bays which are really large tidal basins. The estuary of the Columbia River above Sand Island for approximately 25 miles ranges in width from approximately 3 miles to 6 miles, the flow is divided by sand bars into several channels, and the use of control works is considered impractical. Such works would probably seriously upset the long established regimen and adversely affect the tidal prism essential to maintenance of the entrance channel. As to the use of control works in the wide estuary upstream from Sand Island, the survey report on which the project was based stated in part,

> "For a number of years past a channel 300 feet wide as shown on the map has been maintained by dredging, and comparatively little difficulty has been experienced in maintaining such depths as are required for vessels using the port. . . . it would not appear to be good policy to risk disturbing the equilibrium which has been established and perhaps lose benefits of dredging should the channel have to be changed."

This has always been considered a sound policy, and no control works have been built in the estuary above Sand Island.

The method of determining the proper amount of contraction or river width above the estuary as previously described is simple. It may be called the

practical approach to the problem since none of the various parameters affecting the channel section are specifically or individually considered. All have automatically been applied however, and all their effects integrated by the natural flow of the river over a long period of time in producing and maintaining the desired section.

The prototype is actually a full scale model which has been operating over the centuries, accurately and automatically considering the effects of all the forces which are at work in that particular stream. Back of this operation in the natural stream, however, is the fundamental relation that $V = Q/A$.

If a shoal persists in a certain reach, it is a positive indication that the velocity is not high enough to prevent or remove deposits. To increase V and still keep the equation in balance, it is therefore necessary to reduce the area, A, since the volume of flow, Q, can not normally be permanently changed. In the situation under consideration, the only procedure possible for a reduction of A is to reduce the width of the section, to that indicated by the width curve (Fig. 3). A curve of section areas might be used, but this involves initial computations and eventually resolves itself into use of the width dimension.

The exact value of the mean or the critical velocity necessary to move the material in any case is not known or computed, and the engineer is not specifically concerned with this. It is only necessary that the section be such as to produce the critical velocity (whatever that may be) necessary to prevent deposits, and maintain the depth required, as has been indicated at natural sections in the prototype.

IMPROVEMENT EFFECTED BY CHANNEL CONTROL DIKES

Channel control dikes have been constructed in the river at 16 bars from the foot of Puget Island to Vancouver, Wash. Extensive improvements have been effected, as will be evident from examination of the hydrographic charts, United States Coast and Geodetic Survey (USC and GS) Charts 6151, 6152, 6153 and 6154, as corrected frequently from U. S. Engineers detailed surveys. Several sections and charts are given in Figs. 4, 5, 6, 7, and 8.

Dredging records for most of the bars indicate the reductions in excavation effected. In some cases, however, the records do not reflect the true situation as to excavation necessary, one case in particular being Wauna Bar, where the reported excavation includes also that on the Driscoll range connecting upstream from Wauna Bar, which accounts for nearly all the excavation reported under "Wauna Bar." The Driscoll range is not yet fully controlled. Little if any dredging is now required on Wauna Bar proper (Figs. 7 and 8).

The following tabulations show the reductions in volume of annual dredging at three characteristic bars above the estuary:

At Henrici Bar

Under 30 ft by 300 ft Project:	
1912 to 1919 without control,	
Average excavation per annum	
(On 3 seasons was over 1,000,000)	692,340 cu yd
Under 35 ft by 500 ft Project:	
1946-55 average annual	224,965
1956-60 average annual	162,095
Reduction in excavation even with	
larger channel, per annum	530,245 cu yd

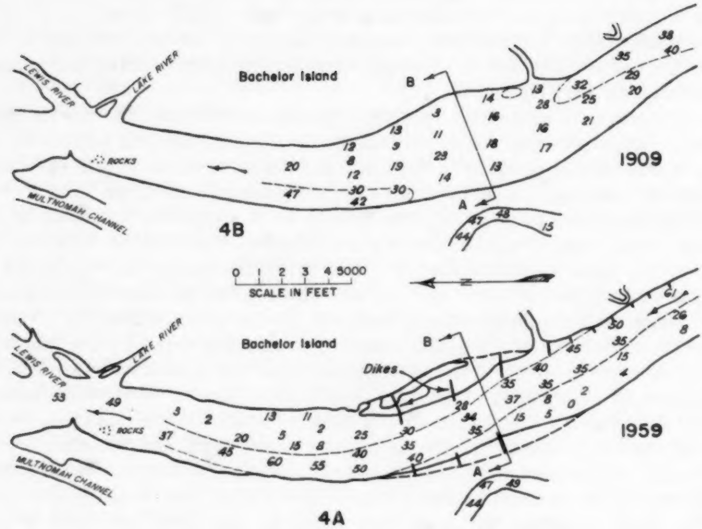

FIG. 4.—HENRICI BAR - 1909 AND 1959

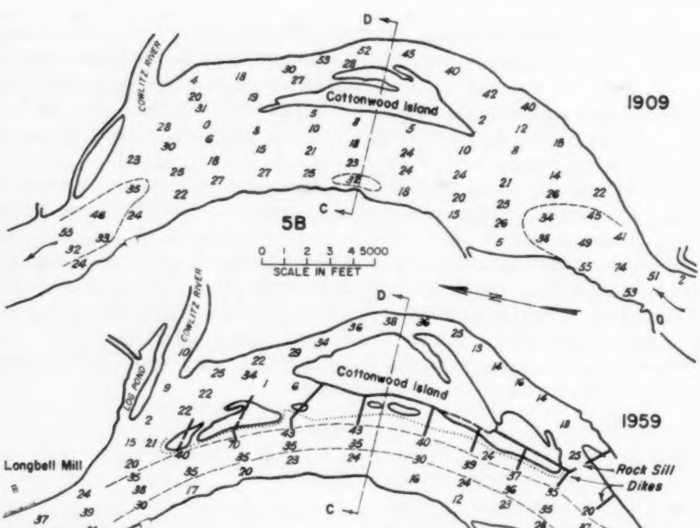

FIG. 5.—DOBELBOWER BAR - 1909 AND 1959

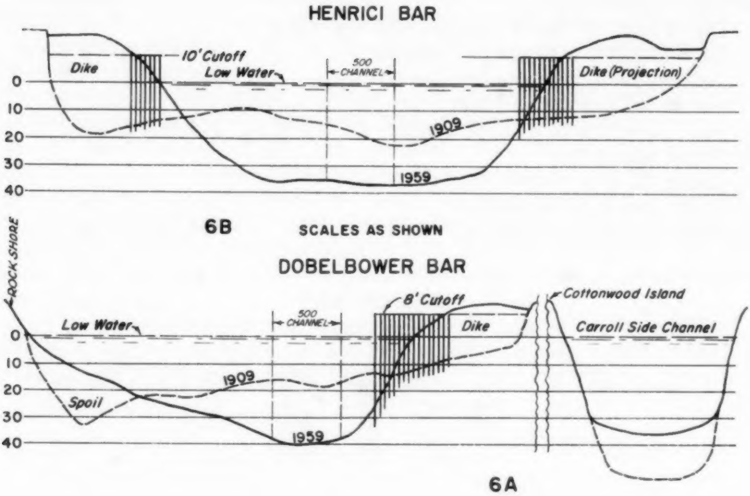

FIG. 6.—CROSS SECTIONS - HENRICI AND DOBELBOWER BARS 1909 AND 1959

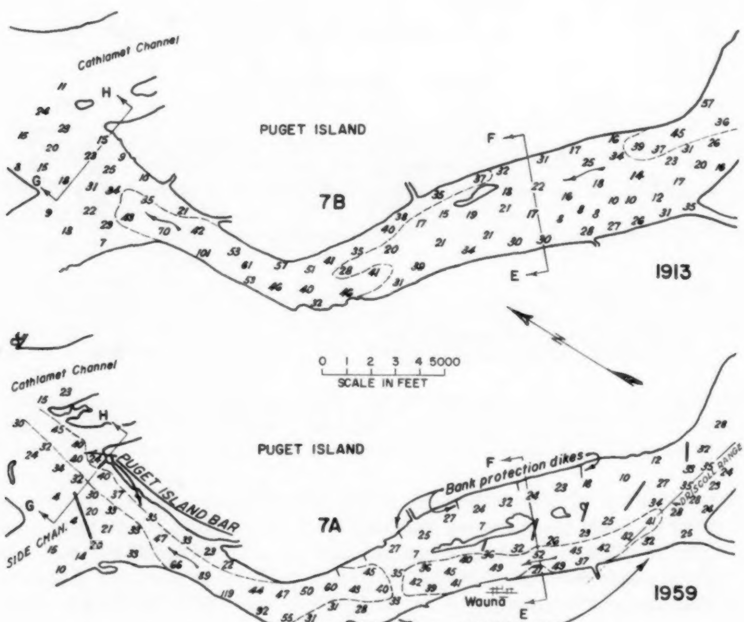

FIG. 7.—WAUNA AND PUGET ISLAND BARS - 1913 AND 1959

At Dobelbower Bar:

Under 35 ft by 500 ft Project with control,	
1946 to 1955, average per annum	919,472 cu yd
1956 to 1960, average per annum	555,000
Reduction per annum, last 5 yr	
over previous 10 yr, per annum	364,472 cu yd

At Puget Island Bar:

Under 30 ft by 300 ft Project	
First dike built in 1925,	
1920 to 1925, average per annum	138,932 cu yd

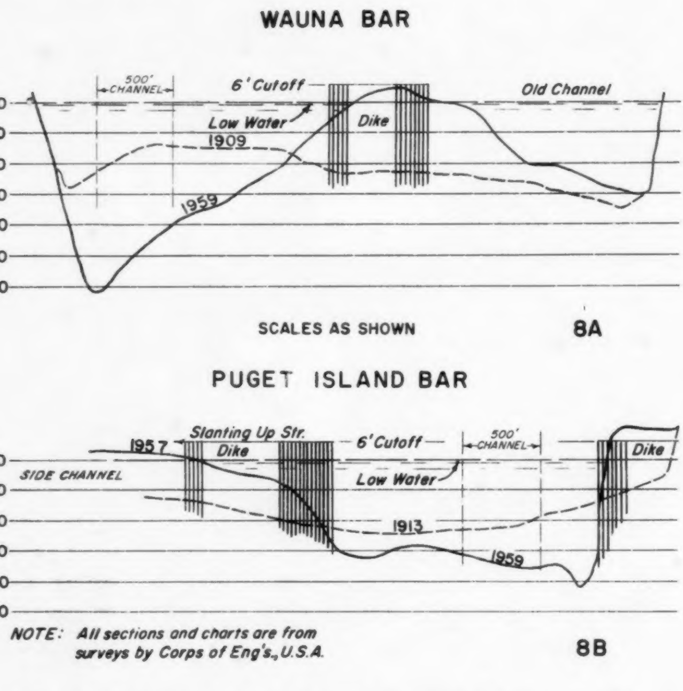

FIG. 8.—CROSS SECTIONS - WAUNA AND PUGET ISLAND BARS - 1909 TO 1959

Under 35 ft by 500 ft Project, controlled:	
1933 to 1945, average per annum	100,473 cu yd
1951 to 1960, average per annum	26,832 cu yd
Reduction even with a larger	
channel, per annum	112,100 cu yd

Henrici and Dobelbower bars were among the hardest on the river to maintain before being put under control. Including normal overdepth, approximately 1,000,000 cu yd were dredged from Henrici on three different seasons under

the 30 ft project before control, and depths after summer freshets were only 18 ft to 20 ft. In recent years little material lies above grade and least depths are approximately 32 ft. Similar improvement is evident at the other bars under control.

The general over-all effect of channel control dikes has been to lower the bed of the river over the various bars. The total amount of material removed by scour since 1920, outside the dredged cuts, is roughtly estimated at 140,000,000 cu yd. This includes also the stretch of river above Vancouver to Bonneville where control dikes have also been in place since 1938. Surveys of all the bars made annually show that the improvement has been continuous and is still in progress, although equilibrium is being approached on many of the bars. The material lying above grade after the freshets is generally in the crests of large sand waves.

While the estimated scour in 40 yr is large in total, it is taking place at a rate of only approximately 3,500,000 per annum. This slow rate is desirable for permanence, and it is not sufficient to upset the regimen of the river. There

TABLE 4.—ANNUAL DREDGING ON ESTUARY BARS FOR FISCAL YEARS 1951 TO 1960

Year	Cubic Yards	Year	Cubic Yards
1951	2,598,032	1956	1,965,321
1952	1,912,256	1957[a]	3,575,126
1953	2,772,621	1958[a]	1,941,045
1954	1,606,849	1959[a]	1,326,589
1955	2,654,848	1960[a]	1,861,509
Totals	11,544,606		10,659,590

[a] Overdepth dredging to depths of 40 ft to 42 ft was done on some of the bars during these years. All material is disposed of in nearby areas.

does not appear to be any evidence of this material being deposited in the estuary, as the records of dredging over the past 10 yr will show. Yardage from all nine bars in the estuary, below and including Skamokawa Bar, is given in Table 4.

These figures show that up-river control works with their resulting scouring effect have not adversely affected the estuary. There is actually a reduction in dredging notwithstanding that dredging on all these estuary bars was done to greater depths in the later years.

It accordingly is apparent that the material scoured from the controlled channel plus that coming down from up-river sources, and material carried in suspension has all passed out to sea, except that probably some of it has to be removed by dredging from Clatsop Spit at the mouth of the Columbia where the channel has shifted out of line and widened to a larger section in recent years.

In his endorsement of the survey report for the 35 ft by 500 ft project the North Pacific Division Engineer stated,[6] that:

> . . . "the success of methods which have been followed in securing and maintaining a channel with a minimum depth of 30 ft at mean lower low water, and 300 ft wide, has been such that there appears to be little question as to the feasibility of securing, by an extension of the same methods of improvement, a channel of materially greater dimensions at reasonable cost.
>
> "The work already done has so improved conditions that the present estimate of yardage to be removed to secure a channel 35 x 500 feet, is less by some 15,000,000 yards than that made six and one half years ago. . . ."

MOUTH OF THE COLUMBIA

Under the original project (30 ft) the south jetty $4\frac{1}{4}$ miles long was completed to the knuckle in 1894. This increased the depth on the bar to 30 ft, but the channel shifted to the north, quickly shoaled again, and the bar advanced seaward. Depths on the bar were then less than 24 ft, and the channel location shifted in cycles from south to north.

A board of Army Engineers was then appointed to report on the situation. They recommended extension of the south jetty $2\frac{1}{2}$ miles and construction of a north jetty to terminate 2 miles north of the end of the extended south jetty, to secure a depth of 40 ft at low water and a width of at least $\frac{1}{2}$ mile.

Extension of the south jetty was completed in August 1913, and the north jetty in 1917. Fig. 9(a) shows condition in 1920.

All the existing works at the mouth of the Columbia, including dikes on Sand Island, Jetty A, as well as the two main jetties, were constructed under the 40 ft project.

Dredging on the outer bar range was also done by the hopper dredge Chinook for several years up to 1918 when the effects of the jetties made such work unnecessary for a channel 40 ft deep by $\frac{1}{2}$ mile wide.

The project was increased, September 3, 1954 to a depth of 48 ft at low water for the same width, $\frac{1}{2}$ mile. This present increased project depth is to be accomplished by dredging and the construction of another jetty (B) between Jetty A and the north jetty if maintenance by dredging is found to be ineffective (Fig. 9(b).

The width of two miles between ends of the main jetties was decided on by the board of engineers in 1904 from observation of the width of the existing natural channel at the throat or gorge section approximately 3 miles to 4 miles upstream. This has proved to be a good decision, as the depth between ends of the jetties has always been satisfactory (40 ft to 50 ft for width of $\frac{1}{2}$ mile or more).[2]

Inside the jetties the channel was in excellent condition with depths of more than 50 ft and good alinement eastward past the easterly end of Sand Island from approximately 1916 to 1932 (16 yr).[7] During this period of good

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6 House Document 195, 70th Congress, First Session, October 26, 1928, p. 48.

7 Discussion by R. E. Hickson of "Interim Consideration of the Columbia River Entrance," by John B. Lockett, Proceedings, ASCE, Vol. 85, No. HY 8, August, 1959, p. 95, Chart A.

channel conditions, however, natural erosion of the north bank in the concave bend along Sand Island and Peacock Spit (which were not then protected or under control) allowed the channel to shift gradually to the north. The south jetty at and near the shore end also deteriorated from ocean wave action from about 1925 to 1932 so that flood tides and wave action carried large volumes

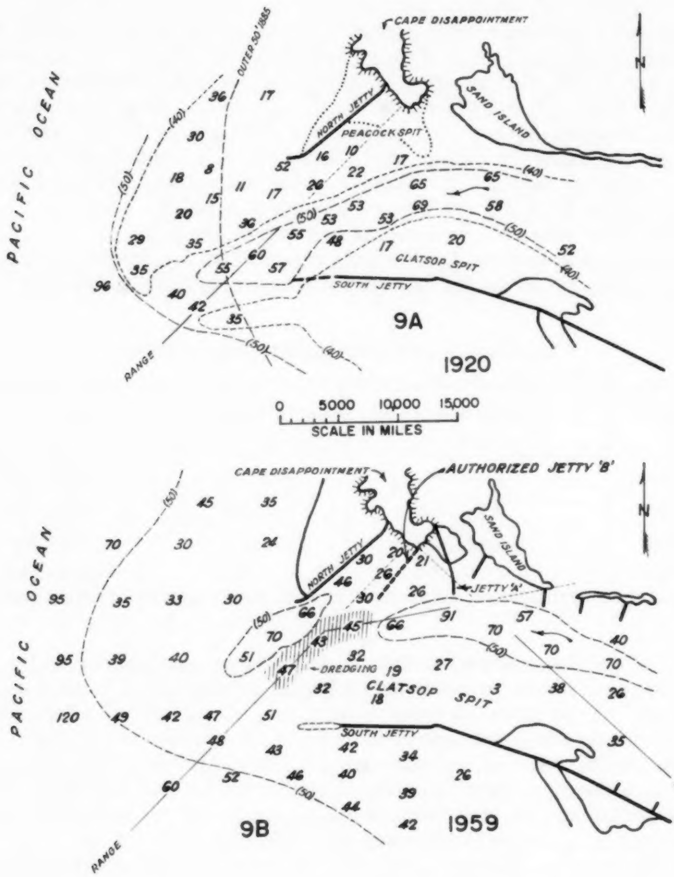

FIG. 9.—MOUTH COLUMBIA RIVER, 1920 TO 1959

of sand from the beach and surf area across and into the south side of the channel, thus aggravating the situation and accelerating the erosion of the north bank and channel shift in that direction, and at the same time adding to the volume of Clatsop Spit.

The south jetty was then restored to cut off the flow of sand; dikes or groins were built along Sand Island, and Jetty A was constructed. The purpose of these

works was, and is, to stabilize the north bank and prevent further shifting of the channel to the north. They have had beneficial effects. Opposite Jetty A the face of Clatsop Spit has been cut back approximately 800 ft at the 50 ft depth, thus improving the channel alinement, and maintaining a channel depth of more than 50 ft for a distance of about $1\frac{1}{4}$ miles downstream from the jetty. Beyond the effect of this jetty (A), however, the ebb currents still swing into the concave bend and allow the formation and persistence of Clatsop Spit on the opposite or south side of the channel, where dredging is now done annually for maintenance of depths and channel alinement, before mentioned (Fig. 9).

Dredging Under the 48 ft Project.—New York dredging at the mouth of Columbia under the 48 ft project, 14,000,000 cu yd, was completed in the fall of 1956, the large U. S. hopper dredge Essayons, and the Biddle being used on this work. Shoaling was rapid during the succeeding winter, and in 1957 maintenance dredging on Catsop Spit amounted to 3,600,000 cu yd. But this covered a width of only 1,500 ft instead of the full project width of $\frac{1}{2}$ mile. In subsequent years dredging was also on a width of 1,500 ft.

Dredging over the past four seasons is given in Table 5.

If dredging has been done for the full project width the volumes would have been greatly increased as the heaviest excavation is along the southerly side

TABLE 5.—RECORD OF RECENT DREDGING

Year (1)	Clatsop Spit, cu yd (2)	Outer Bar cu yd (3)	Total cu yd (4)
1957	3,600,000	400,000	4,000,000
1958	2,290,000	314,000	2,604,000
1959	2,157,000	136,000	2,293,000
1960	2,283,000	2,800	2,286,000

of the cut. It may be assumed, therefore, that annual maintenance on the inner bar (Clatsop Spit) for full project width of $\frac{1}{2}$ mile would amount to approximately 4,000,000 cu yd under present conditions. Maintenance of the outer bar channel presently does not appear to be much of a problem. Shifting the disposal area to the west as recommended by Committee on Tidal Hydraulics placed it in a position where a northerly set of the littoral and tidal currents and wave action do not tend to move the material into the outer bar channel. Dredging on the outer bar in 1960 was only 2,800 cu yd.

Maintenance of the channel on the outer bar is not considered in this paper, but since the bar has not advanced seaward on the line of the outer range for many years, it may be a long time before material trouble will be experienced in this area.[8]

Considering the large amount of dredging necessary for channel maintenance at Clatsop Spit, and the widening of the section to the north in the past 30 yr or more making for poor alinement and shoaling, it is evident that the solution calls for the construction of control jetty B from the north cape as authorized.

[8] "Changes in Columbia River," by R. E. Hickson, Military Engineer, July, August, 1922.

The end of this jetty would be about half-way between Jetty A and the main north jetty, which would place it at the downstream limit of the effects of jetty A. The distance from Jetty A to the end of the main north jetty is approximately $2\frac{1}{2}$ miles.

A jetty at this location, and for the purpose intended, would need to be high enough only for construction purposes, and to cut off the flow into this bend, or in other words to restore the effect of the sand bar which served to control the currents and maintain excellent channel depths for 16 yr (1916-32) before it eroded. It would reduce the section for ebb flow, thus increasing velocities in the channel and to the south, and result in producing a channel section similar to that being maintained opposite and below Jetty A, without dredging. The same principle of channel section control which has been used so successfully at other points on the river is applicable at this location.

This jetty would be located to start near the easterly tip of Cape Disappointment and head directly into the prevailing seas, rather than on some other site or direction. This site will make the jetty much easier to build and also to maintain.

Since alternate plans must be compared as to ultimate annual cost, the estimates for the jetty control plan, and for dredging alone, have been made.

COMPARATIVE COSTS

Plan I, Jetty B Plus Initial Dredging.—

I Jetty B, 14 ft above zero, 25 ft top width, 5000 ft long +		
Stone 1,400,000 tons:		
Cost of stone at \$4.00 per ton	\$5,600,000	
Overhead and contingencies 20%	1,120,000	
Total first cost	\$6,720,000	
Annual costs:		
Interest and amortization at 4%		\$268,800
Jetty maintenance per annum		60,200
Total per annum		\$329,000
Maintenance dredging (initial):		
Est. 2 yr at \$400,000 - \$800,000		
2 yr at 200,000 - 400,000		
1 yr at 100,000 - 100,000		
5 yr at 50,000 - 250,000		
Thereafter 000 - 000		
\$1,550,000		
Average annual dredging for 50-yr life		31,000
I - Total per annum on 50-yr life		\$360,000

Plan II, Maintenance Dredging Alone.—

Dredging alone for $\frac{1}{2}$ mile Channel		
Annual dredging - 1,500 ft channel	\$400,000±	
Annual dredging - 2,640 ft channel	\$700,000	700,000
Savings per annum, Plan I over Plan II		\$340,000
Total savings in life of the project would be large.		

Maintenance work on the jetty would need to be done only at intervals of several years as it is not essential that it be maintained at a high level. Half

to high tide level will be sufficient. Its only function is to keep currents from running into the bend, thus increasing velocities in the channel and improving alinement.

The stone in Cape Disappointment near the root of the jetty, while not of high quality, would be satisfactory for the purpose and could be placed in the jetty at a relatively low cost. The estimate is high enough, however, to cover better quality stone which would be used at the outer end for approximately 200 ft and 100 ft wide, to provide extra material at the exposed outer end for sloughing and deterioration due to wave action. The use of the local rock for the body of the structure, rather than obtaining better stone from a distant source, will result in a large saving in original cost and maintenance.

Some of the Cape Disappointment stone was used on maintenance of Jetty A in 1952 at a cost of $2.89 per ton in place, and some from a nearby quarry also was used during construction of the north jetty about 1915.

From the foregoing estimates and records of dredging in the past 4 yrs on a 1,500 ft channel, it is evident that a tremendous saving in maintenance cost of the project channel at Clatsop Spit just inside the jetties can be effected by the construction and maintenance of the authorized Jetty B. While the Jetty B project, Plan I, shows large savings over Plan II (for dredging alone), this is not the whole saving effected under Plan I. This plan for permanent control will result in providing full project dimensions throughout the year while the channel dimensions obtained by annual dredging alone do not last throughout the winter season, when they are most needed. In other words, while costs are compared on the basis of full project dimensions, Plan II does not actually provide these dimensions at all times. The money value of this deficiency is not estimated in the comparison shown.

As to assurance of the success of the authorized jetty plan, it is necessary only to observe the great improvements, and stability of channels which have been effected by control works at upriver locations, and in the immediate vicinity, and to look at charts showing the excellent channel conditions which prevailed for more than 16 yr (1916-32) in this vicinity without any dredging as long as there was a controlling sand bar on the north side of the channel at site of the authorized control Jetty B. The records are clear in this case.

CONCLUSIONS

As indicated, the theoretical solution for design of channel dimensions involves many variables and combinations of variables, for which values are difficult to determine. How these factors with their various combinations, some favorable, some adverse, can be combined and entered in a workable mathematical solution to determine the dimensions of a self-maintaining channel of the desired depth and width presents a problem for which there apparently is at present no positive or dependable answer.

Fortunately, as previously described in detail, there is a practical and reliable solution which does not attempt to evaluate the effects of each of the parameters or assign any specific values to the various factors or bombinations. It is based on the fundamental relation that $V = Q/A$.

If a shoal persists in a certain reach or area, it is a positive indication that the velocity is not high enough to prevent or remove deposits. To increase the velocity, it is necessary only to reduce the area. Practically, this can be accomplished only by reducing the width.

One or more natural sections in the vicinity or perhaps within a few miles, where satisfactory channel dimensions obtain, can generally be found from which the width between effective banks can be measured. Controlling to this width will solve the problem at the shoal section by producing the critical velocity necessary for section equilibrium at the depth desired.

Nature is the great integrator of the effects of all the forces at work and shows definitely in the prototype the width required in any case. All the forces at work, both favorable and adverse, have been applied in the natural flow, and other conditions with all their variations, over a long period of time. In this simple practical approach to the problem of improvement of a river such as the Columbia, the answer is definitely indicated in the stream itself, a full-scale model which has been operating over the centuries.

In conclusion, the works of improvement and maintenance of the ship channel of the lower Columbia, as outlined herein have been entirely successful, and the methods used may, with confidence, be used on improvement of river channels in general.

ACKNOWLEDGMENTS

The writer appreciates the assistance of many engineers and others of the Corps of Engineers in the Portland Oregon District for their cooperation in securing much of the data contained in the public records, which were needed for completion of this paper.

ADDITIONAL REFERENCES

Additional references concerning works of improvement of the Columbia River are listed.

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2. "Jetty Maintenance at Mouth of Columbia," by R. E. Hickson, Military Engineer, September, October, 1930.
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Proceedings of the American Society of Civil Engineers

CORROSION OF STEEL PILES IN SALT WATER

By James R. Ayers,[1] F. ASCE and Ralph C. Stokes[2]

SYNOPSIS

A description of a field survey of the steel sheet piles in bulkheads at eight widely scattered Naval Stations is presented. Outlined are the reasons for the survey, the method pursued in obtaining samples of the piling, and a brief description of the rates of corrosion for each station. The location, elapsed time since construction of each bulkhead, and the percentage of remaining piling thickness in each zone of exposure are given. The methods of protection used by the Navy both for steel bearing piles and for steel sheet piling are included.

INTRODUCTION

Ferrous metals in the form of cast or wrought iron piles and cylinders were used in waterfront structures beginning about 1850 and ending around 1900. With the rapid rise in the production of steel after 1900, steel sections superseded the use of cast or wrought iron. The first record of the installation of steel sections was about the year 1890. During the period between 1910 and 1918, pier and wharf installations utilizing steel piles and steel frame superstructures became popular. Due to lack of maintenance, these structures soon became corroded above the low water level to such an extent that they were beyond economical repair. As a result, the extensive use of steel went into dis-

Note.—Discussion open until January 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 3, August, 1961.

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favor as a construction material along the waterfront. Thus, in the early history of these structures, the engineering profession became acutely aware of the seriousness of the corrosion problem.

Prior to and during World War II, the Navy became a large user of steel piling because of the relative ease of transporting, handling and driving this material, especially at distant locations. Protection was not provided for the piling in most cases. Since the war, corrosion difficulties in varying degrees have been experienced.

REASONS FOR SURVEY

In recent time, cathodic protection has been put forward as a solution to the corrosion problem. In order to obtain data regarding the variation of corrosion rate from top to a level below mud line for a typical steel sheet pile, the Bureau of Yards and Docks initiated an engineering investigation through the United States Naval Civil Engineering Laboratory, Port Hueneme, Calif., for surveying piling at eight representative Naval Activities. A primary objective of this investigation was to determine whether cathodic protection would be of benefit to the areas that fail first and under what conditions its use should be adopted. Carl V. Brouillette was in direct charge of the field survey, assisted by Alfred E. Hanna. Much of the material for this paper was taken from their report.

METHOD OF SURVEY

Plans for the field survey were predicated on obtaining samples of piling at widely scattered activities chosen to show the effects of differences in maximum and minimum annual temperature, amount of fouling attachment, depth of water, tidal ranges, protective coatings, and evidence of harbor pollution. An analysis of the samples, data from reports received from other Naval Stations, and the results of a literature survey form the basis for the conclusions of this paper.

The field investigation was divided into two phases. In the first phase, a preliminary questionnaire was circulated to the eight Navy facilities, and in phase 2, the actual cutting of the test samples was performed. The purpose of the questionnaire was to provide data and background information prior to taking field samples. At each facility, three widely separated representative piles were selected for cutting samples with torches from the atmospheric, the tidal, the underwater, and below the mud line zones. In order to illustrate the samples obtained during the survey, Fig. 1 shows a series of front views of representative rough-cut sections of sheet piling taken at Coco Solo, which location experienced moderate corrosion. Fig. 2 shows the series of back views of the same rough-cut sections.

The samples were later given an acid bath to clean them and were cut to circular shape of uniform diameter to permit accurate determination of weight loss and residual thickness.

Fig. 3 shows a series of front views of the finished Coco Solo samples and Fig. 4 is a series of edge views. Fig. 5 shows edge views of the finished samples from Alameda. These were in the best general condition considering the 20-yr length of exposure. The San Diego samples shown in Fig. 6 were prob-

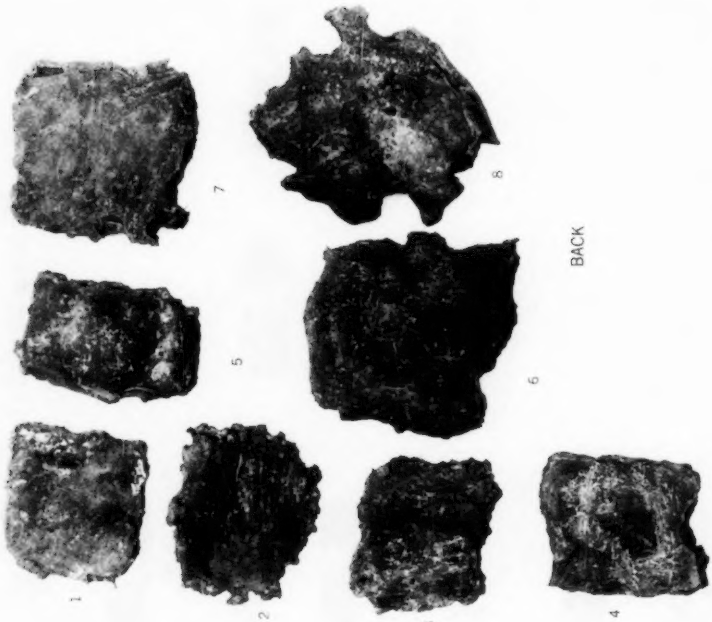

FIG. 2.—BACK VIEWS OF ROUGH-CUT SECTIONS OF SHEET PILING

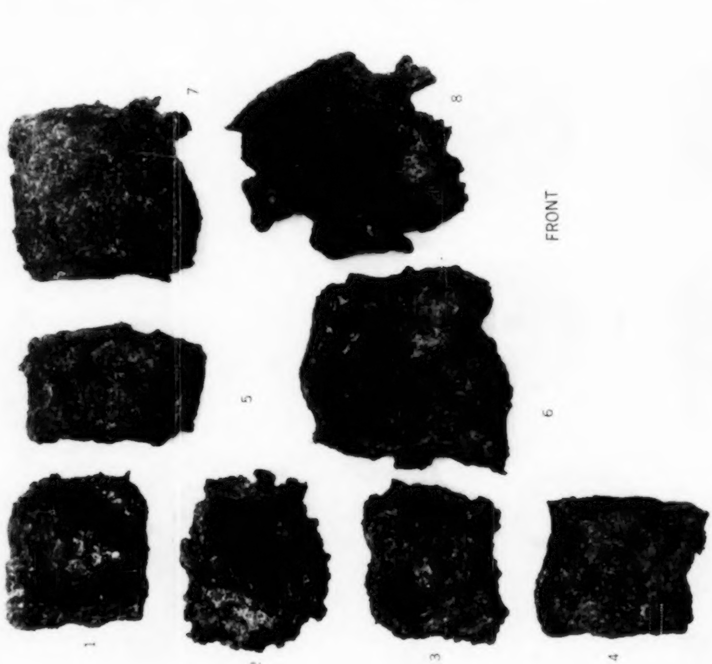

FIG. 1.—FRONT VIEWS OF ROUGH-CUT SECTIONS OF SHEET PILING

FIG. 3.—FRONT VIEWS OF FINISHED SAMPLES

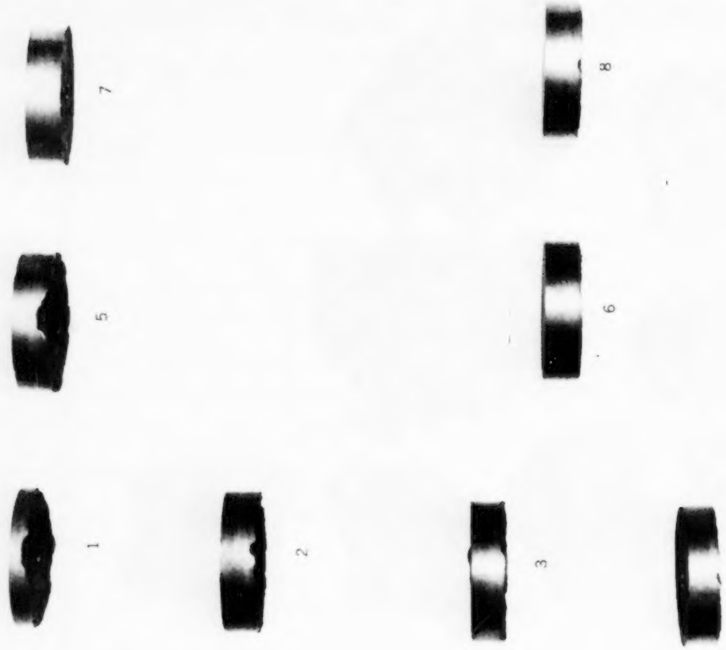

FIG. 4.—EDGE VIEWS OF FINISHED SAMPLES

FIG. 5.—EDGE VIEWS OF FINISHED SAMPLES FROM ALEMEDA

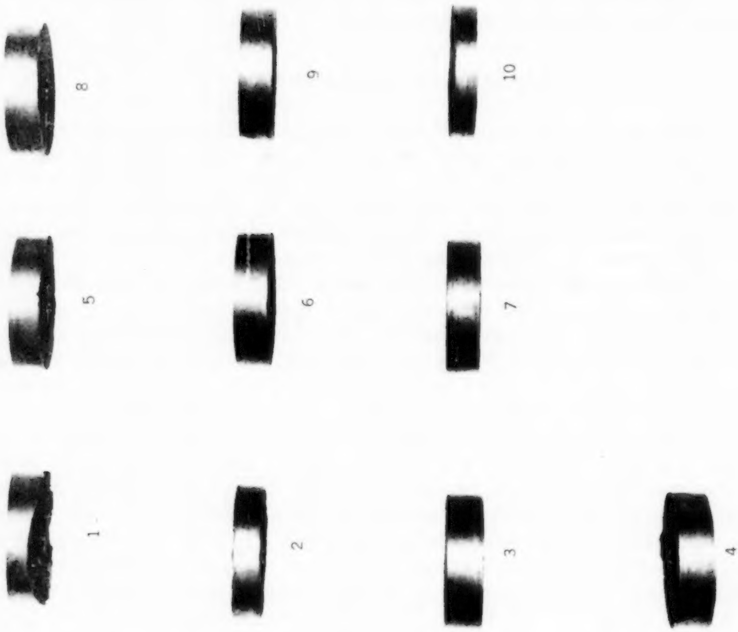

FIG. 6.—SAN DIEGO SAMPLES

ably influenced by pollution at the bottom of the bay and by the prevailing wave action on the upper portion of the sheet piles.

RESULTS OF THE SURVEY

The vertical distribution of corrosion rates for the East Coast Stations are shown on Fig. 7. A brief description of the rates for each station follows:

Boston Naval Shipyard, Boston, Mass.—The samples were taken from a bulkhead in shallow water of 5 ft maximum depth with 10 ft tide range. The maximum corrosion rate observed was 8.2 mpy at about 1 ft above mean low water. The next highest rate was 7.9 mpy at the mud line.

The maximum average corrosion rates were 5.2 mpy at 2 ft below MLW and 4.6 mpy at the mud line. The minimum average rate was 1.0 mpy at mid-tide and at 2 ft above mean high water (MHW). The maximum average thickness of original pile remaining after 17 yr were 95% at 2 ft above MHW, and 94% at mid tide and at 1 ft above the mud line; the minimum averages remaining were 75% at 2 ft below MLW and 80% at the mud line and just above MLW.

Norfolk Naval Shipyard, Portsmouth, Va.—The maximum corrosion to the steel sheet piling at Norfolk was about 17 mpy in the anodic area 2 ft below MLW. The maximum average rate was 7.8 mpy at 2 ft below MLW. The average corrosion at half water depth and below varied from 2 to 2.5 mpy. Originally, a bitumastic coating was applied to these piles during construction in 1933. This coating extended from the top of the piling to a distance of about 1 ft or 2 ft below MLW, but only a very slight amount was in evidence on the piling near the mean low water line. Weight loss measurements showed that 58% of the piling remains in the area 2 ft below MLW after 26 yr. At half depth, 84% remains; elsewhere, approximately 90% or more remains.

Key West Naval Station, Key West, Fla.—The piling surveyed was coated full length prior to being driven. The maximum corrosion rate, 19 mpy, occurred in the splash zone 2 ft above MHW. Another area of high corrosion rate, 10.3 mpy, occurred at 2 ft below MLW. The maximum average rate was 10 mpy at the top whereas the minimum rate was 1.5 mpy at half tide. The average piling weight remaining in the atmospheric zone after 21 yr was 54%; in the splash zone, 68%; at half tide, 94%; at 2 ft below MLW, 83%; and about 88% at lower levels.

U. S. Naval Station, Coco Solo, Canal Zone.—The maximum corrosion rate, based on the thickness of the thinnest portion of the test samples, was 17.3 mpy in the splash zone about 2 ft above MHW. From 2 ft below MLW downward, the maximum corrosion rate decreased rather uniformly form 8.8 mpy to 4.0 mpy at 1 ft below the mud line. The maximum average corrosion rate was 10 mpy in the splash zone; 2.5 mpy at half tide; 5 mpy at 2 ft below MLW; and somewhat less at half depth. After 24 yr of exposure an average of about 51% of the steel in the piling remains in the splash zone, 71% at half tide, 74% at 2 ft below MLW, 77% at half depth and higher percentages below.

Fig. 8 shows the vertical distribution of corrosion rates for West Coast and Pacific Sections.

Puget Sound Naval Shipyard, Bremerton, Wash.—The steel sheet piling sampled at Puget Sound was driven in 1946 without a protective coating. A cathodic protection system using graphite anodes was applied in 1954 and was checked and adjusted every two months since that date. The maximum corrosion rate

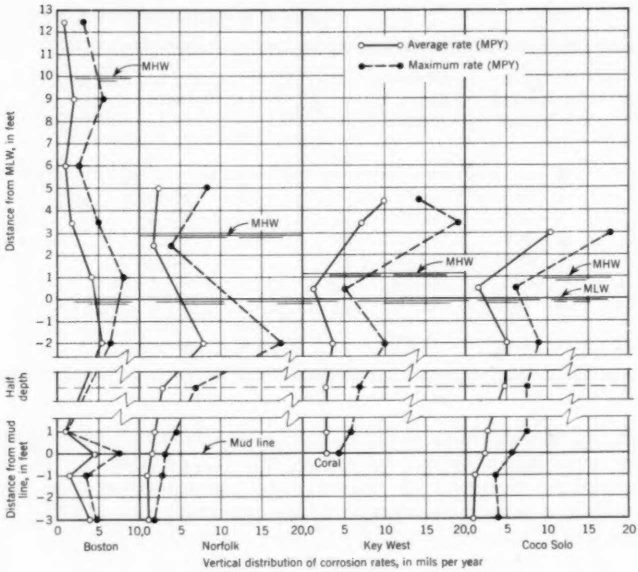

FIG. 7.—VERTICAL DISTRIBUTION OF CORROSION RATES FOR EAST COAST STATIONS

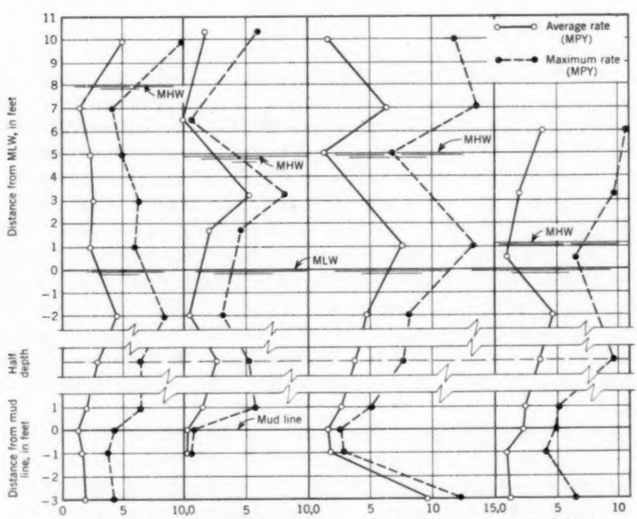

FIG. 8.—VERTICAL DISTRIBUTION OF CORROSION RATES FOR WEST COAST AND PACIFIC STATIONS

TABLE 1.—PERCENTAGE OF REMAINING

Location	Tide Range, in feet	Elapsed Time, in Years	Atmos-pheric	Splash	Elevation	
					+9	+7
Boston	10	17		95	89	94
Norfolk	2.8	27		90		
Key West	1.3	21	54	68		
Coco Solo	0.9	24		52		
Bremerton	10	13		87		96
Alameda	4.7	20	91	100		
San Diego	4.9	17	93	72		
Pearl Harbor	1.2	13	86	93		

is 10 mpy in the splash zone. Cathodic protection does not extend to this area. A maximum rate of 8.5 mpy was shown at 2 ft below MLW. The maximum average rate was 5.0 mpy in the splash zone and somewhat less below MLW, corresponding to a minimum average remaining thickness of 87%. The average remaining thickness was 92% to 96% elsewhere after 13 yr.

U. S. Naval Air Station, Alameda, Calif.—The maximum rate of corrosion, 8.6 mpy, occurred in the tidal range approximately 2/3 of the way from MLW to MHW. The maximum average rate of corrosion in this area was 5.6 mpy. At half depth the average rate was 2.5 mpy. After 20 yr the thinnest section had about 84% of the piling remaining. At half depth, more than 88% remained. The low corrosion rate at Alameda can be attributed to a film of fuel oil continuously coating the piling in the upper area.

U. S. Naval Station, San Diego, Calif.—The harbor in this area is reported to receive discharge from sewage treatment plants and has little regular change of water other than that caused by tidal action. The maximum rate of corrosion, 14 mpy, occurred at approximately 1 ft above MLW and again at about 2 ft above MHW in the splash zone. The maximum average rate of corrosion was about 10 mpy at a level 3 ft below the mud line. Elsewhere, the average rates were 7 mpy at 2 ft above MHW and 8 mpy at 1 ft above MLW. The minimum thickness remaining was 65% of the original at 3 ft below the mud line after 17 yr. The corrosion rate here is much greater than that measured below the mud line at any of the other facilities surveyed. This higher rate can only be explained by a lower pH in this zone or an oxygen concentration cell. The effects produced by the sewage disposal effluent in the basin and the slight stagnation in this area were not analyzed. At 1 ft above mean low water, the average remaining thickness is 70%; above high water 72%, and is considerably higher elsewhere.

Pearl Harbor Naval Shipyard, Pearl Harbor, Hawaii.—The piling from which samples were taken had been coated to full length with one coat of bitumastic material prior to being driven in 1946. No evidence of the initial protective coating remained above half depth. Considerable rusting was observed in the spalsh and atmoshpheric zones. The maximum corrosion rate of 11 mpy was found near the top of the piling about 6 ft above MHW. Two additional areas with a severe corrosion rate, 10 mpy, were at 2 ft above MHW and at half depth.

PILING THICKNESS BY ZONES

Tidal			Immersed		Mud Line			
in feet above MLW					Levels in feet above Mud Line			
+5	+3	+1	Elevation -2	Half Depth	+1	0	-1	-3
	89	81	75		94	78	93	82
		91	58	84	90	91	94	94
		94	83	89	87	87		
		88	74	77	88	89	94	94
93	92	94	88	92	94	96	95	95
	84	90	92	89	93	99	98	
	92	70	78	81	83	94	93	65
		96	88	90	94	93	97	96

The corrosion of one specimen taken at half depth had one large pit. This condition could have been caused by damage to the protective coating followed by accelerated corrosion in the small anodic area. After the major portion of the coating became detached from the piling in the area surrounding the pit, the corrosion became more general and less localized. The maximum average corrosion rate was 4.8 mpy at 2 ft below MLW. The minimum average was 1.0 mpy at half tide and below the mud line. The minimum average thickness remaining was 85% at the top after 13 yr.

Table 1 summarizes the findings of the survey as to the average percentages of remaining thickness found in the various zones throughout the height of the piling as determined by the weight loss of the samples.

RESULTS OF THE QUESTIONNAIRE

The data obtained from the questionnaire indicates that the mean warm water temperature ranged from 87°F at Key West to 50°F at Bremerton, and the mean cold water temperature varied from 35°F at Boston to 80°F at Coco Solo. Water temperatures were taken at varying depths. The water temperature near the surface would be expected to be higher than the temperature at greater depths, especially during the summer months. Thus, the corrosion rate in the tidal and atmospheric zones would be affected more by the ambient air temperature than by the deep water temperature. The fouling appears to be heavier in the areas of warmer year-round temperatures, but in some areas both fouling and corrosion in the tidal zone were considerably reduced by floating oil resulting from spillage and small boat activity in the harbor.

RESULTS OF LITERATURE SURVEY

A review of technical publications indicated that that highest rate of corrosion to steel piling used in waterfront structures is generally in the region of periodic wetting by sea water. Experimentally measured corrosion rates during periodic wetting increased with concentration of chloride ion up to 8% by weight of sodium chloride in water. Above this concentration, the corrosion rate de-

creases because of the reduction in solubility of oxygen in the solution. During periodic wetting in sea water, thin films of electrolytic solution saturated with oxygen are formed on the surface of the piling throughout the tidal zone. Between this wet area above water and the area below water, corrosion cells are produced by the different oxygen concentrations. The area of the piling in the relatively oxygen poor sea water is anodic to the exposed area, and the rate of corrosion in this area is reported to be very intense because of the short lines of current flow produced in the electrolytic corrosion cell. The high rate of corrosion is maintained by the continual change in the anodic and cathodic areas during raising and lowering of the tide; the high concentration of oxygen in the thin film of sea water causes rapid depolarization in this area.

Solution potential measurements made on uncoupled steel specimens showed that the plates in the tidal zone were strongly cathodic with respect to the plates constantly under water. Thus, when electrically coupled, the continuously immersed specimen would act as a positive electrode for current flow to the steel specimen in the tidal zone.

A differential aeration cell also occurs at the mud line because of the lower oxygen content below the mud line (anodic area). The intensity of corrosion in this area is less than that occurring in the tidal area.

As sea water temperature increases, fouling growth rates increase. The corrosion rate of unprotected steel also increases with the increase in sea water temperature. However, excess fouling protects the steel and reduces the tendency toward increased corrosion.

Cathodic protection of steel piling requires electrical continuity. The resistance of sea water is so low that it is an ideal conductor for cathodic current. In the tidal area, electrical conductivity between an anode and the steel piling occurs only where continuity exists between the surface film of sea water on the piling and the main body of sea water. Because the piling in the tidal area are moistened by the fluctuating tide, and impressed current will only protect about one-half of the area between high and low water mark.

The literature survey indicates a preference for protecting steel piling in sea water environment by a combination of a protective coating and cathodic protection. It is usually advantageous to coat the entire pile before driving it so as to lower the current requirement. Even a defective coating is better than none at all.

Protection of steel piling in the tidal and atmospheric zones is the major problem in piling protection. The British found that cathodically protecting steel piling in the absence of a protective coating causes calcareous deposits to form and extend about two-thirds of the way up the inter-tidal range. No corrosion of the piles was observed in the region covered by the deposit. They also report that steel piling can be completely protected by over-lapping coating and cathodic protection. They use a quick drying coating, applied during periods of low tide, to protect the piling above mid tide.

Because the formation of alkali at the cathode is an inevitable feature of cathodic protection in sea water, the presence of saponifiable components in the protective paint coating should be avoided.

To insure the flow of electrical current between the piles they must all be electrically continuous. During construction it is possible to electrically bond the piles through the reinforcing mats of the decking concrete or by use of welded steel bars between piles.

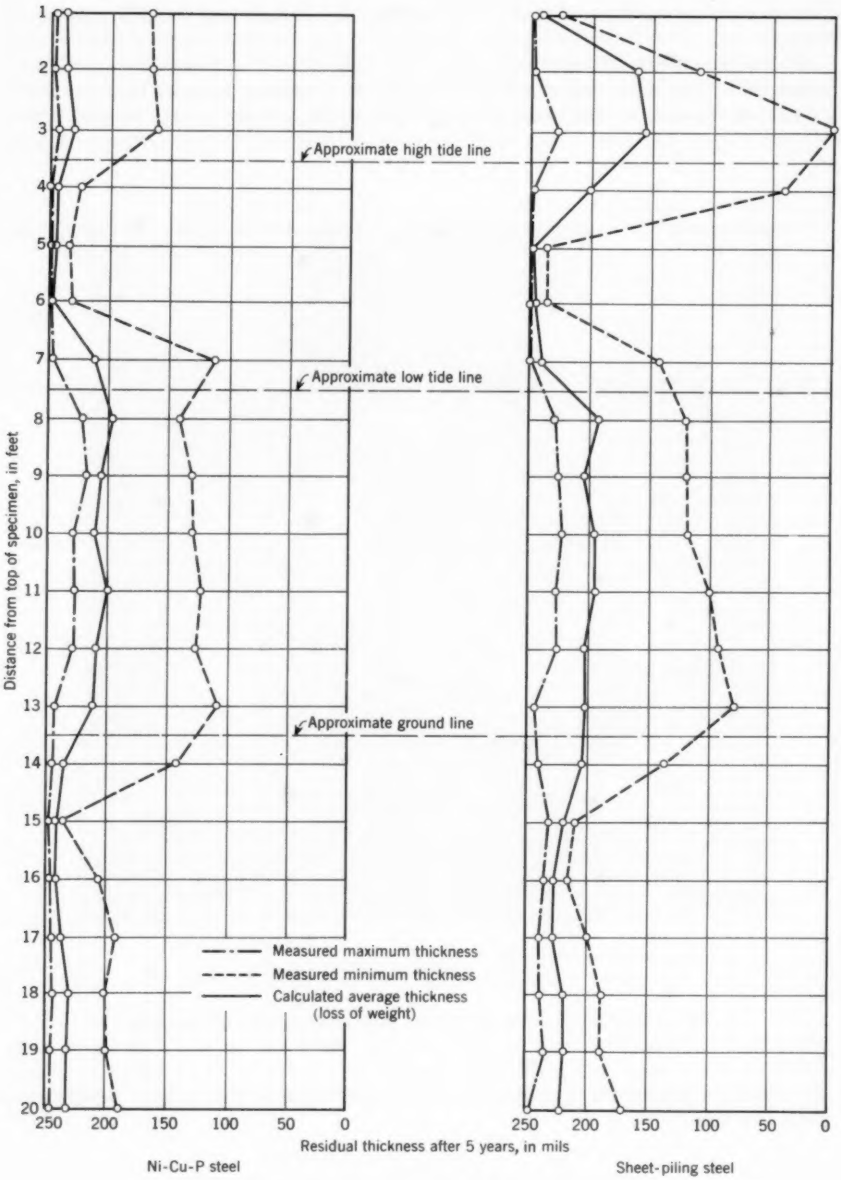

FIG. 9.—COMPARATIVE CORROSION ON TWO STEELS IN MARINE ENVIRONMENTS

In the absence of cathodic protection, breaks in coatings covering the continuously submerged portion of piling subject the local areas to severe galvanic action.

Protection of the piling above mean low tide greatly reduces the sacrificial corrosion in the area below mean low tide. If a coating is used to protect the upper area, breaks in the coating would expose only a very small cathodic area

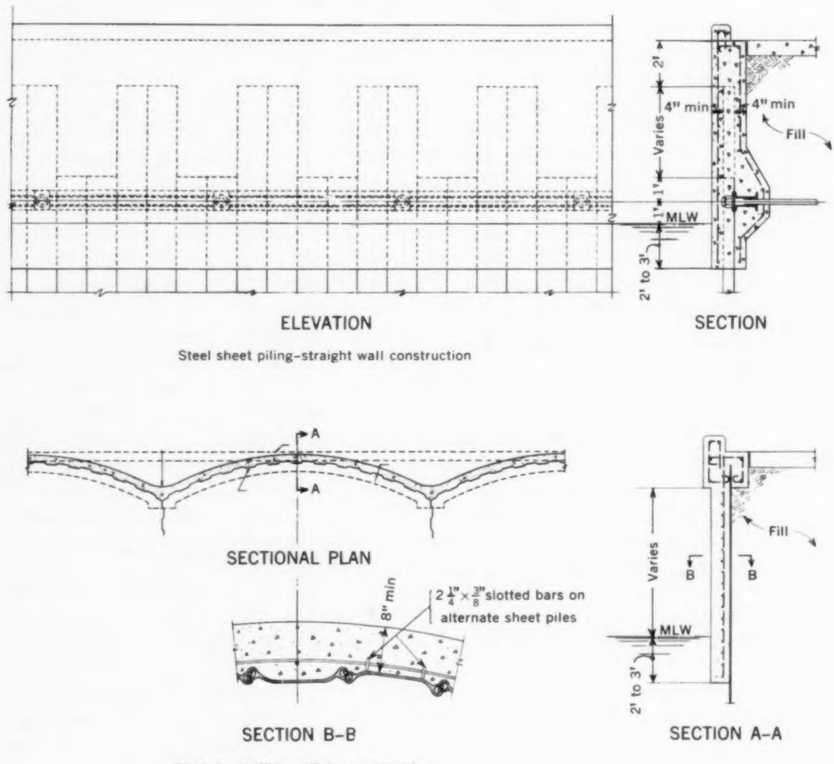

FIG. 10.—CONCRETE JACKETS FOR STEEL SHEET PILING

and would not cause any appreciable accelerated attack on the bare steel below mean low tide.

CORROSION OF MULTI-ALLOY STEEL

Studies are being directed toward an improved alloy steel to replace the steel presently used for fabricating steel piles. Fig. 9 shows the effects of

corrosion on Nickel-Copper-Phosphorus steel plates as compared to ordinary Carbon steel plates. This illustration is taken from a paper[3] by C. P. Larrabee. The three curves on each of the two illustrations give the measured maximum and minimum and computed average residual thickness of plates for an exposure period of 5 yr. As can be noted, the dotted curve for the Carbon Steel indicates a corrosion rate considerably more rapid than that shown by the dotted curve for the Nickel-Copper-Phosphorus Steel above half tide, whereas below half tide level, there is very little difference between these two

FIG. 11.—EXTENT OF CORROSION ON H-PILING

types of steel. During the test, the area of the Carbon Steel exposed in the splash zone lost five times as much weight as a similarly exposed area of the alloy steel.

METHODS OF PROTECTION

The Navy has tried many types of protective coatings for steel piling. These coatings have not proven effective within the tide range.

[3] "Corrosion-Resistant Experimental Steels for Marine Applications," by C. P. Larabee.

The concrete jacketing of steel piling has proven very effective when it extends from a level well below MLW to the top of the piling above high tide. Fig. 10 shows jackets for two types of steel sheet pile construction. Both straight wall and cellular type construction are illustrated. It will be noted that, in the interest of economy, alternative pairs of sheet piles in the straight wall construction are indicated in shorter lengths to save steel. For cellular construction, the concrete facing is formed on the outer face only, because the rate of rusting on the inner face is relatively low if the cells are filled with select material.

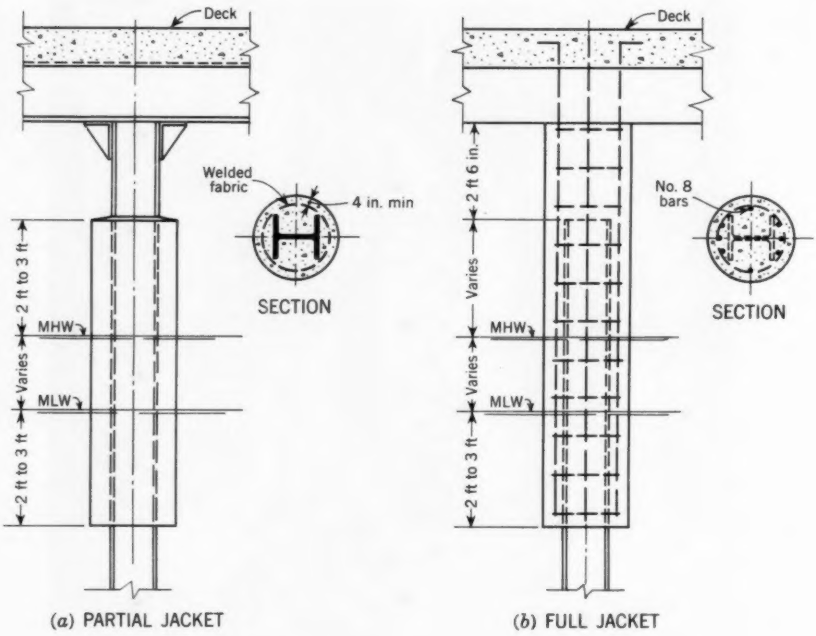

FIG. 12.—CONCRETE JACKETS FOR STEEL H PILES

Fig. 11 shows the extent of corrosion above MLW on H-piling supporting mooring platforms located at Guantanamo Bay, Cuba. These piles were repaired by cutting out a section of the pile from elevation minus 2 to plus 4, splicing in a new section of H-pile and encasing in a concrete jacket applied by a combination of casting and guniting. The immersed portion of these piles was of sufficient thickness after 15 yr to last another 15 yr period without endangering the stability of the superstructures.

Fig. 12 shows two methods of providing protection for H-piling in the critical tide zone. A partial jacket is shown in Detail A. A full jacket is indicated on Detail B, in which the H-pile does not extend up to the concrete superstruc-

ture. Instead, the load is transferred through bond from the concrete column section to the H-pile.

CONCLUSIONS

1. The corrosion rate of steel piling based on all available information is summarized as follows:

a. The maximum average rate occurs above and/or within the tide zone.

b. A secondary maximum average rate of lesser magnitude occurs in the anodic area about 2 ft below mean low water.

c. The average rate at half depth is comparatively low.

d. The average rate in the vicinity of the mud line is generally quite low.

e. High rates do not generally occur on the landward side of sheet piling except in the case of porous backfills with water filtering down behind the piling. High rates are experienced on the landward side when coral fills are used in tropical climates.

2. A properly maintained cathodic protection system reduces the rate of corrosion below the mid tide level.

3. A protective coating over the full length of a pile reduces the corrosion rate as long as the coating remains. Deterioration of the coating occurs first in the tide zone and progresses upward to the top, and at a slower rate below water. On most Navy installations, failures of coatings in the tide zone have occurred within the first 2 yr.

4. The most complete protection system for steel piling is concrete jacketing in the tide range and cathodic protection below mid tide.

5. Multi-alloy steel has a low corrosion rate for the upper portion of piling. It offers an alternative that may be comparable to concrete jacketing in longevity. Furthermore, the superior physical properties of multi-alloy steel will permit a much greater reduction of thickness below water without exceeding the allowable design stresses.

6. Cylindrical steel bearing piles are the most desirable shape for unprotected piling because of the smaller external area exposed to corrosion.

7. An adequate and satisfactory procedure for protecting steel piling in water-front structures is as follows:

a. Initial incasement of the upper portion of all steel piles in dense concrete jackets.

b. When the lower portions of the jackets require renewal, the thickness of the submerged portion of the piling should be measured to determine whether cathodic protection is required to extend the useful life of the submerged portion beyond that of the repaired jackets.

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HARBOR PARADOX

By John Miles[1] and Walter Munk[2]

SYNOPSIS

The resonant frequencies and their "Q" (or sharpness) have been derived for rectangular harbors characterized by a length d, a width b, and entrance width a, assuming $b \ll \lambda$ in which λ is the incident wave length. The seiche amplitude varies as $Q^{1/2}$. It is found that Q increases with increasing b/a and d/a, thus leading to the paradoxical result that a narrowing of the harbor mouth (relative to the other dimensions) diminishes the protection from seiching.

INTRODUCTION

Many harbors are limited in their usefulness not so much by the penetrating sea and swell as by surging (or seiching) associated with the harbor resonances. There can be no question that the prominent surge frequencies are those of the gravest normal modes, but this does not tell much as to the nature of the excitation. The modes can be excited by stresses exerted on the water surface within the harbor. There are documented instances where the surging followed the passage of an atmospheric pressure jump, and others where a sudden shift in the winds appears to have been responsible. These are examples of a systematic variation in the normal and tangential stresses on the water surface. Random variations occur at all times and also must contribute to the surging. Of incidental interest is the excitation of the nor-

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mal modes by movements of the harbor bottom associated with the arrival of earthquake waves.

Seiches can also be excited through the harbor entrance. If there is a broad spectrum of waves exterior to the harbor, then those particular frequencies that correspond to the resonant frequencies will excite the interior resonances. Typical resonant periods are of the order of minutes. It appears that this mechanism of seiche generation has received relatively little attention, probably because such low-frequency waves in the open sea had not been observed. But in recent years the ocean wave spectrum has been found to extend continuously from swell to tides, and in some localities the spectrum is even peaked at frequencies typical of harbor seiches. There can be no question that severe seiching associated with the arrival of tsunamis (tidal waves) is excited through the harbor mouth.

In the following, the writers cite that excitation through the mouth is the predominant mechanism. To what extent this is the case for any given harbor could be established by a few definitive measurements which (to the writers' knowledge) have never been conducted.

Notation.—The letter symbols used in this paper are defined where they first appear in the text and are arranged alphabetically, for convenience of reference, in the Appendix.

Statement of Paradox.—Assuming, then, that seiches excited through the harbor entrance are the limiting factor in the usefulness of a harbor, a narrowing of the entrance leads not (as one might expect) to a reduction in harbor surging, but to an enhancement. This is called the harbor paradox.

The Q of a Harbor; General Considerations.—The analysis can proceed a considerable way without detailed mathematical treatment. In this section the general considerations are developed, and these are applied under the heading, Excitation Through the Harbor Mouth, to the observed exterior wave spectra off California and Mexico. The principles underlying the mathematical treatment and the results are given in the section, Direct Computation of Q. Details of the derivation are contained in the sections on the Boundary-Value Problem and the Rectangular Harbor.

The response of a single-degree-of-freedom oscillator at the frequency f may be described by the normalized impedance

$$Z = 1 - \left(\frac{f}{f_o}\right)^2 + i\,Q^{-1}\left(\frac{f}{f_o}\right) \qquad (1)$$

in which f_o is the resonant frequency and $1/Q$ is a linear measure of the damping ($Q = 1/2$ would correspond to critical damping). The power-amplification factor of the oscillator is defined by

$$A^2(f) = \frac{1}{|Z|^2} = \frac{1}{\left[1 - \left(\frac{f}{f_o}\right)^2\right]^2 + Q^{-2}\left(\frac{f}{f_o}\right)^2} \qquad (2)$$

and is shown in Fig. 1. It increases from unity at $f = 0$ to a maximum of Q^2 at $f = f_o$ and then decreases monotonically to zero as $f \to \infty$. Assume $Q >> 1$ throughout this study. The half-power points then are given by $f = f_o\left(1 \pm \frac{1}{2} Q^{-1}\right)$, and the relative bandwidth, defined as the difference between these two frequencies divided by f_o, is simply $1/Q$. Also approximate the amplification factor in the neighborhood of the resonant frequency by

$$A^2 \doteq \frac{1}{4\left[1 - \left(\frac{f}{f_o}\right)\right]^2 + Q^{-2}}, \quad f \doteq f_o \quad \ldots\ldots\ldots\ldots (3)$$

or, equivalently,

$$\frac{A_o^2}{A^2} = 1 + 4\,Q^2\left[1 - \left(\frac{f}{f_o}\right)\right]^2, \quad A_o = A\,(f_o) \quad \ldots\ldots\ldots\ldots (4)$$

The amplification factor of a many-degree-of-freedom system, such as a harbor, is naturally more complicated than that given by Eq. 2, but if the resonance of any particular mode is sufficiently sharp, the behavior of the amplification factor in the neighborhood of resonance will be governed by Eq. 4. Accordingly, we define the Q of the harbor by comparing A_o^2/A^2 to Eq. 4.

In the case of a free oscillation in the harbor, the disturbance decays because of a radiative loss of energy through the mouth. The relative loss of

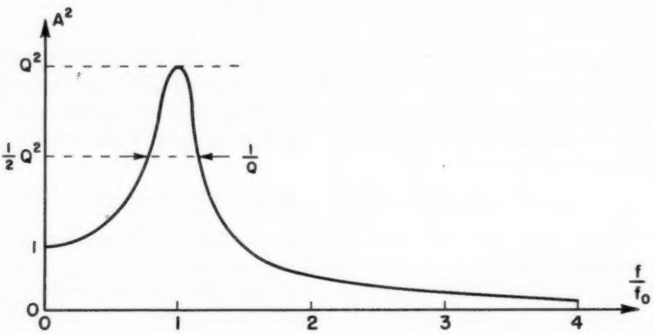

FIG. 1.—POWER AMPLIFICATION FACTOR

energy per cycle is $2\,\pi/Q$. This is equivalent to the previous definition of Q for the case of steady state excitation. Accordingly, the Q increases with decreasing width of the mouth. The nature of the relation between Q and the entrance width depends on the harbor configuration and will be computed for a number of examples. In most cases Q is between 2 and 10.

There are two obvious limitations to the foregoing considerations. One is that it takes time of the order of Q/π cycles for the harbor surges to adjust themselves to any exterior excitation. Thus, if Q were extremely high, the harbor would not respond appreciably to a severe but short lived excitation. Actually this is a minor consideration. Severe excitation is likely to last for 1 day or longer (even in the case of tsunamis), and for a typical harbor this would represent approximately 50 cycles. On the other hand, for Q = 10 the adjustment to the increased excitation is virtually complete in half a dozen cycles, and the harbor has ample time (unfortunately) to adjust to the extreme conditions.

The other consideration is that as the harbor mouth becomes increasingly narrower, the internal dissipation eventually must predominate over the

energy loss through the mouth. At this stage further narrowing does not lead to any appreciable deterioration (nor does it lead to improvement). A rough estimate can be made as follows: The stress on the sea-bed is roughly $C \rho u^2$, in which C is a constant of the order 10^{-3}, ρ denotes density, and u represents velocity. The dissipation per unit time per unit area is then

$$\frac{dE}{dt} = C_\rho \overline{|u^3|} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

If $u = u_o \cos(\omega t)$, then $\overline{|u^3|} = (4/3\,\pi)\, u_o^3$ is the mean-cubed-velocity. In shallow water (depth h) the amplitudes of elevation and velocity are in the ratio

$$\frac{a}{u_o} = \frac{h}{\sqrt{gh}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The mean energy per unit area of a standing wave is $E = \frac{1}{4}\rho g a^2$. Combining all these expressions

$$Q = \frac{\omega E}{\frac{dE}{dt}} = \frac{3\pi}{16\,C} \quad \frac{\omega h^2}{a\sqrt{g\,h}} \quad \ldots\ldots\ldots\ldots\ldots (7)$$

Note that Q depends on amplitude. For small oscillations the "specific dissipation" (essentially Q^{-1}) within the harbor ultimately becomes negligible as compared with radiative losses through the mouth. More generally, add reciprocal Q's associated with different mechanisms of loss insofar as each such reciprocal Q is small. Let

$$h = 10 \text{ meters}, \quad \left(\frac{2\pi}{\omega}\right) = \frac{1}{2} \text{ hr}, \quad a = 1 \text{ meter};$$

then $Q = 20$, so that even for this extreme case the internal dissipation is small as compared with the typical radiative losses.

The dissipation resulting from turbulent flow through the entrance has not been considered. As the entrance becomes increasingly narrower, this eventually may become the limiting factor (Rectangular Harbor). The problem is complicated by non-linear interaction between tidal flow and the flow associated with seiching.

Excitation through the Harbor Mouth.—Now let $S_1(f)$ designate the power spectral density of the excitation, such that $S_1(f)\,\delta f$ designates the contribution towards the mean-square-elevation of the sea surface from a frequency interval $f - \frac{1}{2}\delta f$ to $f + \frac{1}{2}\delta f$. The power spectral density within the harbor is then given by

$$S_2(f) = A^2(f)\; S_1(f) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

and the mean-square harbor oscillation is $\int_0^\infty S_2(f)\,df$. If, as is usually the case, $S_1(f)$ varies only slowly in the neighborhood of f_o, the shape of $S_2(f)$ will approximate that of $A^2(f)$, and the mean-square response will be given approximately by

$$\int_0^\infty S_2(f)\,df \doteq S_1(f_o) \int_0^\infty A^2(f)\,df = \frac{1}{2}\pi\, Q\, f_o\, S_1(f_o) \ldots\ldots (9)$$

It is concluded that the root-mean-square amplitude of the harbor response increases as $Q^{1/2}$.

As an illustration, consider the spectrum off Oceanside, Calif., shown in Fig. 2. The units of the vertical scales are cm^2 per cycles · hr^{-1}; those of the horizontal scale are cycles · hr^{-1}. (The spectra for Acapulco, Mexico and Oceanside, Calif. are shown in Fig. 3. The units are the same as in Fig.

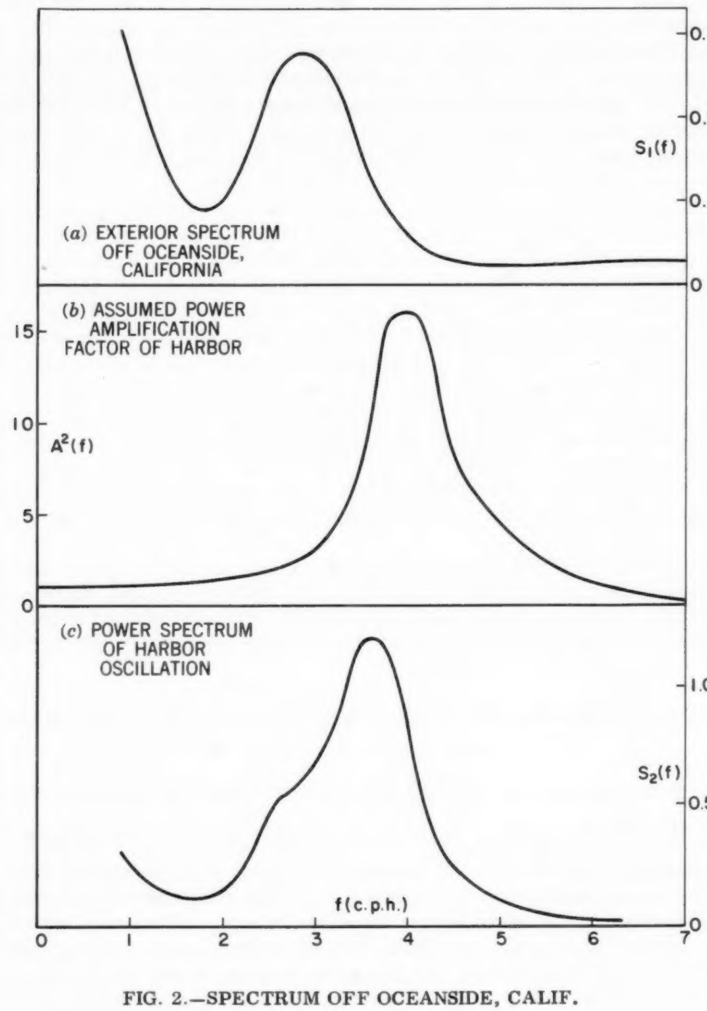

FIG. 2.—SPECTRUM OFF OCEANSIDE, CALIF.

2.) It is peaked at approximately 3 cycles per hr (cph), or at a period of 20 min. The curve is based on measurements of pressure fluctuations along the sea bottom. The spectra were obtained from the time series by means of high-

speed digital computers using the method of Tukey. Details are tedious and uninteresting; they have been adequately described elsewhere.[3] The spectral densities given herein are typical of day-to-day activity in this region. On extreme days the densities may be 10 times the values shown, but the shape of the spectrum at any given locality is remarkably unchanged. On the other hand, tsunamis (tidal waves) may be associated with 10^3 times the plotted values. Again the shape is not dissimilar with that resulting from day-to-day background.

For illustration, a power amplification factor that is peaked at $f_o = 4$ cph (15 min period) and has a Q of 4, as sketched in the center of Fig. 2 was selected. The response of the harbor is plotted at the bottom and represents the product of the two upper curves, in accordance with Eq. 8. The harbor response is peaked at a slightly lower frequency than the resonance frequency of

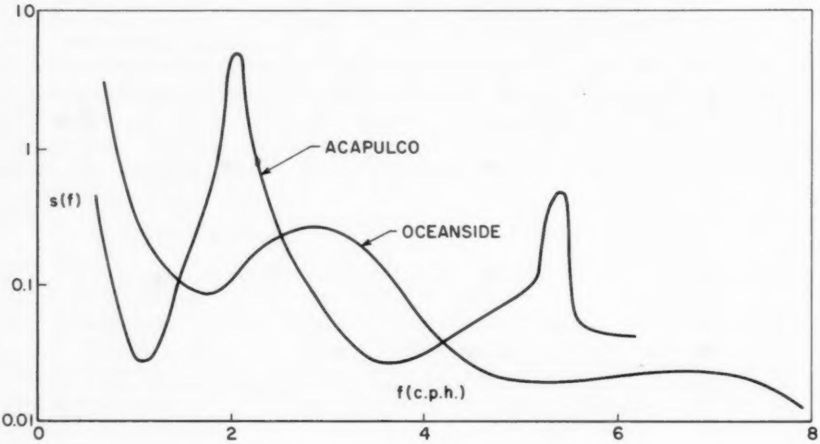

FIG. 3.—SPECTRA FOR ACAPULCO, MEXICO, AND OCEANSIDE, CALIF.

4 cph on account of the negative slope of the exterior spectrum. The area under the bottom curve, $\int_0^\infty S_2(f)\,df$, is roughly 2 cm^2. This means that the mean-square elevation (instantaneous level above mean water level) is 2 cm^2; the root-mean-square elevation is $\sqrt{2}$ cm. The root-mean-square amplitude (wave crests above mean level) is $\sqrt{2}\,\sqrt{2} = 2$ cm. The root-mean-square height (crest to trough) is 4 cm. (Wave heights as reported by visual observers are of the order of 1.5 times the root-mean-square heights.) In the case of tsunamis the energy densities may be 10^3 times higher, and thus the heights of harbor oscillations would be of the order $4\sqrt{1000} = 120$ cm.

3 "Spectra of Low-Frequency Ocean Waves," by W. H. Munk, F. E. Snodgrass, and M. J. Tucker, Bulletin, Scripps Institution of Oceanography, Univ. of California, Vol. 7, 1959, pp. 283-362.

There are then two ways to reduce seiches in harbors: (a) to mismatch the harbor amplification with the exterior spectrum and (b) to reduce the Q of the harbor. The peaks in the exterior spectrum appear to be determined by the dimensions of the continental shelf; those in the amplification curve depend on the depth or the size of the harbor and to some extent on the width of the entrance (Fig. 9). To mismatch the two curves one must alter either the exterior spectrum or the harbor amplification. The latter is easier. In the foregoing example the worst possible condition would correspond to a harbor resonating at $f_0 = 3$ cph. The mean square values would then be 4 times as large, and the amplitudes twice what they are for $f_0 = 4$ cph. If the Q could be reduced from 4 to 2, the reduction in amplitude would be by $\sqrt{2}$.

Which of these two methods is to be more rewarding depends primarily on the shape of the exterior spectrum. In the case of flat, monotonic spectra, the shift in f_0 does not help; when there are extremely sharp peaks, a minor shift in harbor tuning could bring about a noticeable improvement (or deterioration). As an extreme example the spectrum off Acapulco, Mexico,[4] with a spike at 2 cph is presented. The spectral densities are drawn on a logarithmic scale, and the Oceanside spectrum is plotted for comparison. The Acapulco peak is associated with the steep continental slope dropping into the Acapulco Submarine Trench. In general the gentle California spectrum is more representative of off-shore conditions.

In the present paper only one resonant peak of the harbor was considered. The proper procedure for estimating the interior disturbance is to consider all important modes. In general it will be found that the contribution from higher modes diminishes. One reason is that the Q diminishes; another that the exterior spectrum drops off at high frequencies (and this has to do with typical dimensions of the continental shelf as compared with typical dimensions of harbors).

Furthermore, it may be more significant to compute the spectrum of the orbital velocities, rather than of amplitudes, the velocities being primarily responsible for seiche damage. Let u be a typical velocity, h the harbor depth, $c = \sqrt{gh}$ the phase velocity, and z the elevation above mean level. Then since

$$\frac{u}{c} = \frac{z}{h}$$

it follows that the velocity spectrum is of the order

$$\left(\frac{c}{h}\right)^2 S_2(f) = \frac{g}{h} S_2(f),$$

in which $S_2(f)$ is the elevation spectrum previously presented. Note that the velocity and elevation spectra are proportional. In the previous example a mean square elevation of 2 cm^2 was obtained. Let h = 10 meters. Then since $g = 9.8$ m sec^{-2}, the mean square velocity (in $cm^2\ sec^{-2}$) equals numerically the mean square elevation in cm^2. Accordingly the root-mean-square value of peak velocities is 2 cm sec^{-1}. During tsunamis this would be of the order of 60 cm sec^{-1}, or 1.2 knots.

The conclusion is then to mismatch the harbor resonances with the exterior spectra (particularly in regions of sharp exterior peaks) and to design the

4 "Sea Level Spectra, 0.3 to 1.5 Cycles Per Hour, at Acapulco and Salina Cruz," by W. H. Munk and H. Cepeda, Anales del Instituto de Geofisico, Univ. Nac. Auto. de Mexico.

largest possible openings (lowest Q) consistent with adequate protection from sea and swell. An alternative solution is a breakwater against which sea and swell are dissipated but which is "transparent" to the low-frequency oscillations.

Note that the analysis so far has been based on quite general principles. Now consider means of evaluating f_o and Q for a given harbor configuration. There is a large literature on the evaluation of f_o, and there is little one can contribute except to show that the widely published formula for correcting f_o for harbor width is wrong. The evaluation of Q requires a computation of either the radiation from the harbor mouth or the amplification curve in the neighborhood of f_o.

Direct Computation of Q.—Assume long waves in water of uniform depth h. Let

$$z(x, y, t) = R\,l\left\{\zeta(x, y)\, e^{i\omega t}\right\} \qquad (10)$$

be the free-surface displacement

$$c = (g\,h)^{1/2} \qquad (11)$$

the wave speed, and

$$k = \frac{\omega}{c} = \frac{2\,\pi}{\lambda} \qquad (12)$$

the wave number; then[5] ζ satisfies the Helmholtz equation

$$\nabla^2 \zeta + k^2 \zeta = 0 \qquad (13)$$

The particle velocity is given by

$$\underset{\sim}{q}(x, y, t) = R\,l\left\{\left(\frac{i\,g}{\omega}\right) \nabla \zeta(x, y)\, e^{i\omega t}\right\} \qquad (14)$$

Consider a harbor of area S that communicates with the open sea through a mouth M in the plane barrier x = 0, as shown in Fig. 4. Noting that the normal velocity (which is proportional to ζ_x) vanishes everywhere on the barrier, the disturbance radiated from the mouth in terms of the normal velocity there may be expressed according to[6]

$$\zeta(x, y) = \frac{1}{2}\, i \int_M H_o^{(2)}(k\,R)\, \zeta_x(0, \eta)\, d\eta \qquad (15)$$

in which

$$R = \left[x^2 + (y - \eta)^2\right]^{1/2} \qquad (16)$$

is the distance between a point $(0, \eta)$ in the mouth and a point (x, y) in the open sea, $H_o^{(2)}$ is a Hankel function of the second kind, and the η-integration is to be taken only over the mouth. At large distances from the mouth ($kr \gg 1$), $H_o^{(2)}(k\,R)$ may be replaced by its asymptotic approximation and then replace R by the polar radius r in keeping with the assumption that the mouth is small compared with the wavelength; these approximations yield the radially symmetric wave

$$\zeta(x, y) \sim i\,(2\,\pi\,k\,r)^{-\frac{1}{2}}\, e^{-i(kr - \frac{1}{4}\pi)} \int_M \zeta_x(0, \eta)\, d\eta \qquad (17)$$

5 "Hydrodynamics," by H. Lamb, Cambridge Univ. Press, New York, 1932, p. 189.
6 Ibid., p. 305, Eq. 16.

(If the mouth had not been assumed to be small compared with the wavelength, R could have been approximated by $r - \eta \sin\theta$, in consequence of which the additional factor $\exp(ik\,\eta \sin\theta)$ would have appeared in the integrand of Eq. 17). The mean potential and kinetic energies associated with this radiated wave are each equal to $\frac{1}{4}\rho g |\zeta|^2$ per unit area and are propagated across the semi-circle r = constant, $|\theta| < \frac{1}{2}\pi$ at the wave speed c. The total rate at which energy is radiated from the mouth therefore is given by

$$P = \frac{1}{2}\int_{-\frac{1}{2}\pi}^{\frac{1}{2}\pi} \rho g c\, |\zeta|^2 r\, d\theta = \frac{1}{4}\rho g c k^{-1}\left|\int_M \zeta_x(0,\eta)\, d\eta\right|^2 \quad \text{. . (18)}$$

To complete the computation of Q, the total mean energy of the disturbance, for example $\overline{E}$ is required. This, too, will be half potential and half kinetic in

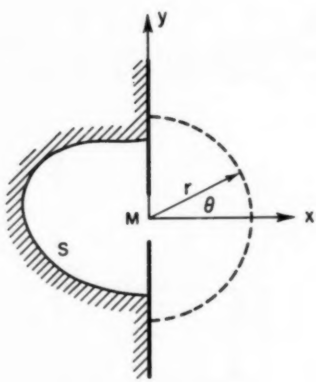

FIG. 4.—HARBOR OF GENERAL SHAPE (S) THAT COMMUNICATES WITH THE OPEN SEA THROUGH A MOUTH (M) IN A PLANE BARRIER

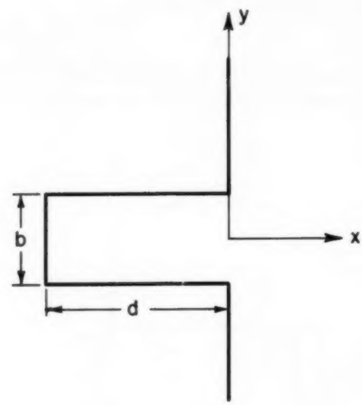

FIG. 5.—OPEN RECTANGULAR HARBOR

the neighborhood of resonance, and the potential energy (but not, in general, the kinetic energy) will be confined almost entirely to the harbor; accordingly

$$\overline{E} = \frac{1}{2}\iint_S \rho g\, |\zeta|^2\, dS \quad \text{. . . . . (19)}$$

Then compute Q according to

$$Q = \frac{\omega\,\overline{E}}{P} = 2\,k^2 \iint_S |\zeta|^2\, dS \Big/ \left|\int_M \zeta_x\, dy\right|^2 \quad \text{. . . . . . . (20)}$$

To proceed further consider a specific harbor and develop an appropriate expression for ζ. The problem is essentially similar to that presented by a two-dimensional acoustic resonator, and the classical techniques[7] are generally applicable. Consider, for example, the rectangular harbor of Fig. 5 on the assumption that

$$d \gg b \quad \text{(21a)}$$

and

$$k\,d = \frac{1}{2}\,\pi \quad \text{(21b)}$$

Eq. 21(b) implying resonance in the gravest mode. Treating the disturbance radiated from the mouth as a small perturbation, then pose the conventional (as for a one-dimensional organ pipe) solution

$$\zeta = C \sin\left(\frac{\pi\,x}{2\,d}\right),\ -d \leqslant x \leqslant 0 \quad \text{(22)}$$

Substituting Eq. 22 in Eq. 20 and carrying out the integrations, the remarkably simple result is obtained as

$$Q = \frac{d}{b} \quad \text{(23)}$$

In the next two sections it will be shown that the approximation of Eq. 23 is rigorously correct for the harbor of Fig. 5 in the limit $b/d \rightarrow 0$. More gen-

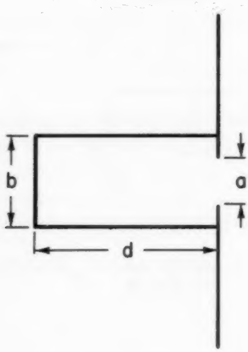

FIG. 6.— RECTANGULAR HARBOR WITH CONSTRICTED, SYMMETRIC MOUTH

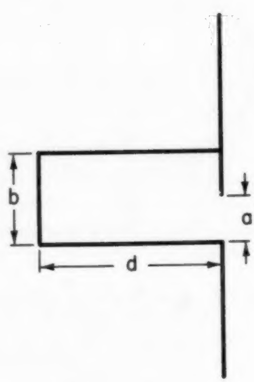

FIG. 7.— RECTANGULAR HARBOR WITH CONSTRICTED, ASYMMETRIC MOUTH

erally, however, the approximation of constant velocity in the mouth is much too rough for the formulation of Eq. 20 and would have been entirely inadequate had the barrier appreciably overlapped the walls of the harbor, as in Figs. 4, 6, and 7. A more general formulation will be developed in the sections on the Boundary-Value Problem and the Rectangular Harbor, which is not only less sensitive to the approximation of ζ, but which also provides an estimate of the effect of the mouth on the resonant frequency.

7 "Theory of Sound," by Lord Rayleigh, Dover Publications, New York, 1945, Chapter 6.

The results of the subsequent analysis for the harbor of Fig. 6 can be summarized in terms of the two equations

$$\cot\left(k_o d\right) = k_o b \left\{0.478 - \frac{1}{\pi}\ \ln\left[\left(k_o b\right)\left(\frac{a}{b}\right)\sin\left(\frac{\pi}{2}\ \frac{a}{b}\right)\right]\right\} \quad \ldots\ldots (24)$$

and

$$Q + \pi^{-1} = \frac{d}{b}\ 2\ \frac{1 + \dfrac{\sin\left(2\ k_o\ d\right)}{2\ k_o\ d}}{1 - \cos\left(2\ k_o\ d\right)} = \frac{\dfrac{d}{b}}{f_1\left(k_o\ d\right)} \quad \ldots\ldots (25)$$

The roots to Eq. 24 determine the resonant wave numbers, k_o, and from these the wave lengths, $\lambda_o = 2\ \pi/k_o$, and frequencies, $f_o = \sqrt{g\,h}/\lambda_o$, are readily

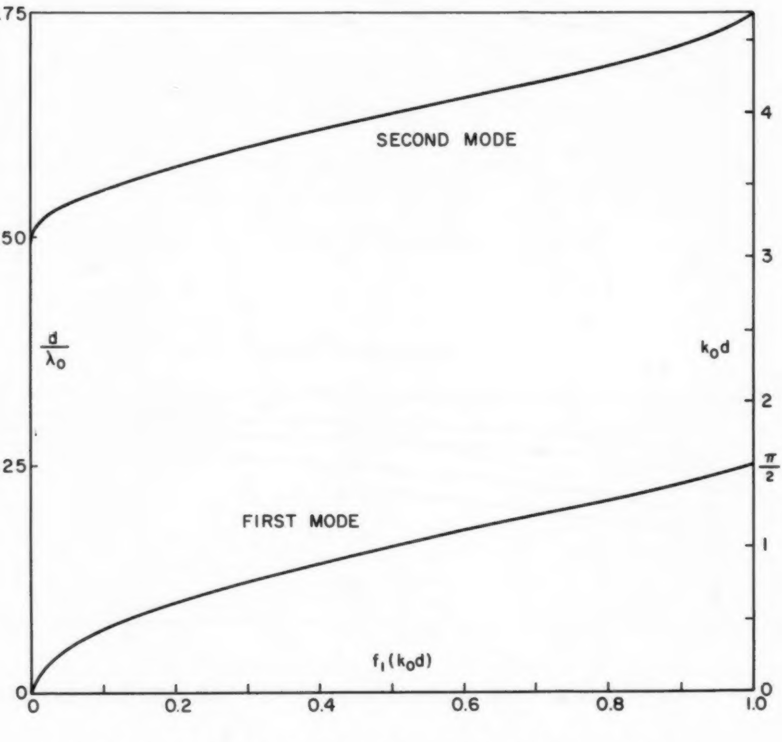

FIG. 8.—PLOT OF THE FUNCTION $f_1\ (k_o d)$ APPEARING IN EQ. 25

derived. For any root the corresponding value of Q then follows from Eq. 25. The function appearing in Eq. 25 is plotted in Fig. 8 for a range at $k_o\,d$ covering the two gravest modes (there are no roots to Eq. 24 for $\frac{1}{2}\ \pi < k_o\,d < \pi$). The results presume $k_o\,b \ll 1$ and are independent of the direction of the incident waves to order $k_o^2\ a^2$.

Fig. 9 is a presentation of these equations for the two gravest modes. The solid curves give the resonant wave length, λ_o, as a function of the entrance width, a, the harbor width, b, and length, d (Fig. 6). The dashed curves give corresponding values of equal Q. All quantities are in dimensionless ratios and satisfy the long-wave approximation, $k_o\, b \ll 1$. The ordinate varies from a closed harbor (a/b = 0) to an open harbor (a/b = 1). In the asymptotic case of an infinitely narrow (b/d = 0) and open (a/b = 1) rectangular harbor the usual "open organ pipe" relations $d/\lambda_o = \frac{1}{4}, \frac{3}{4}, \ldots$, is obtained. Moreover

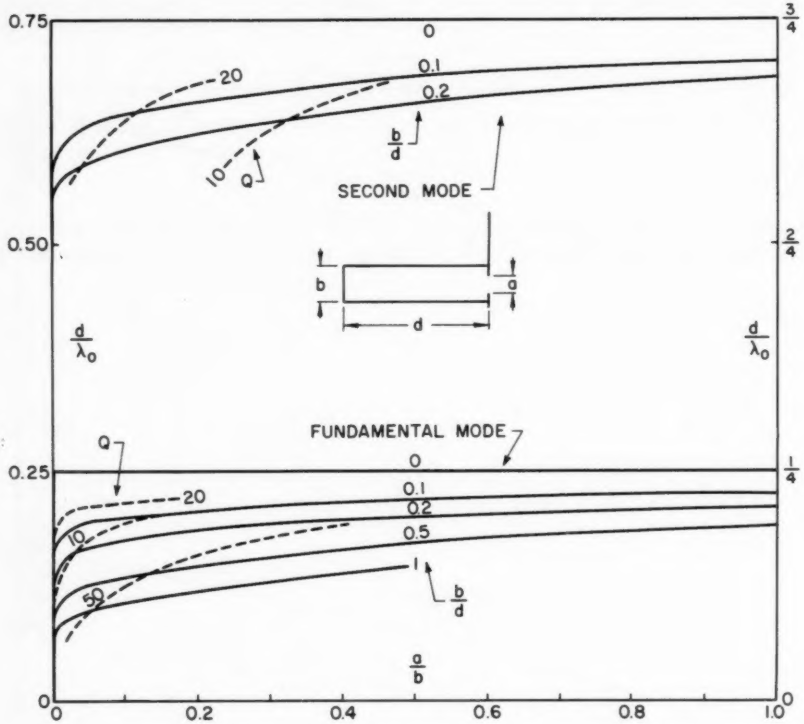

FIG. 9.—PRESENTATION OF EQ. 24 AND EQ. 25 FOR THE TWO GRAVEST MODES

$Q = d/b = \infty$. The appropriate asymptotic formulas for the fundamental mode are

$$\lambda_o = 4\left[d + b\ f_2\left(\frac{b}{d}\right) + b\ f_3\left(\frac{a}{b}\right)\right] \qquad (26)$$

$$f_2\left(\frac{b}{d}\right) = \frac{1}{\pi}\left[1.051 + \ln\left(\frac{d}{b}\right)\right] \qquad (27)$$

and

$$f_3\left(\frac{a}{b}\right) = \frac{1}{\pi}\ \ln\left[\frac{b}{a}\ \csc\left(\frac{\pi}{2}\ \frac{a}{b}\right)\right] \qquad (28)$$

Thus the harbor can be considered as a "quarter-wave-length" resonator provided its length is considered to be enhanced by an amount b $f_2(b/d)$ associated with the aspect ratio b/d plus an amount b $f_3(a/b)$ associated with the

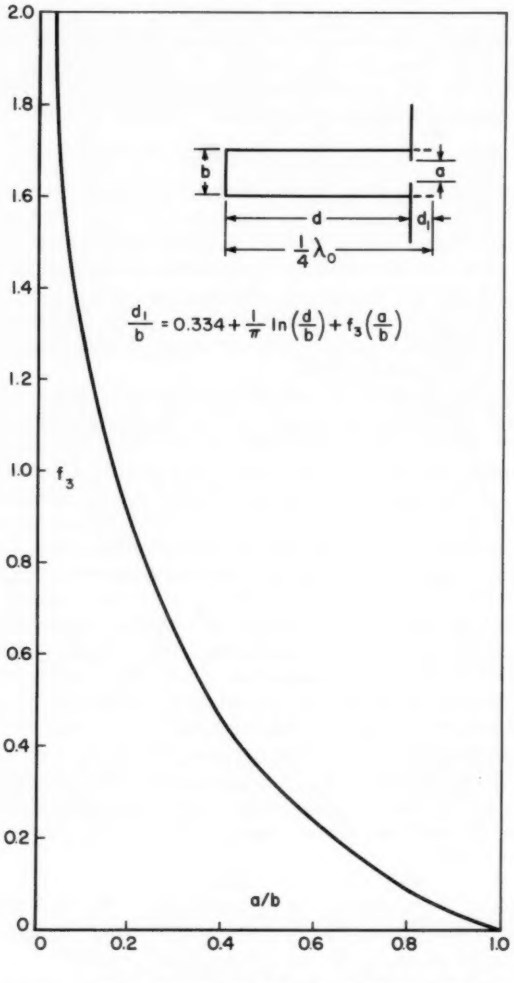

FIG. 10.—END-CORRECTION FOR THE DOMINANT MODE IN THE HARBOR OF FIG. 5 (EQS. 26 TO 28)

aperture ratio a/b. The function $f_3(a/b)$ is plotted in Fig. 10. In textbooks the length correction due to a finite aspect ratio is widely quoted as 0.41 b (rather than π^{-1} b [1.051 + ln (d/b)]) in analogy with Rayleigh's correction for open pipes.

Note that closing the aperture leads to an increase in d/λ_0 and thus to a decrease in resonant frequency. For a closed harbor the gravest mode corresponds to $\lambda_0 = 2$ d. This mode corresponds to the asymptotic descendent of the second mode for the open harbor. The fundamental mode of the open harbor approaches zero frequency with diminishing a/b. On account of the logarithmic form of f_3 the asymptotic values of d/λ_0 are approached only for exceedingly small aperture ratios. For the case b/d = 0.2

	a/b	= 0,	.01	.001
fundamental	d/λ_0	= 0,	0.13	0.15
second mode	d/λ_0	= 0.15,	0.55	0.56

But in the case of openings as narrow as 0.1% or 1% of the harbor width, the turbulence at the entrance must be an important factor. The present solution is based on a variational principle which in effect emphasizes the volume flux through the entrance and is relatively insensitive to the detailed current profile. In the case of narrow entrances, dissipation reduces this flux and would thus appear to make the entrance narrower than it is. The asymptotic values corresponding to a closed harbor therefore must be approached more rapidly than in the present, idealized model.

For the fundamental mode the frequency approaches zero as a/b approaches zero, and the two-dimensional analog of the Helmholtz resonator[8] results. Such a resonator consists of a vessel that is nearly closed and communicates with the surrounding atmosphere only through a small neck or aperture. The wave length in the dominant mode of oscillation then is large compared with the dimensions of the vessel, and the motion of the air is in approximately the same phase everywhere but in the neighborhood of the neck, or aperture. In the analogous resonance of a harbor, the displacement would be approximately independent of location throughout the interior of the harbor; the potential energy of the motion would be confined primarily to the interior; and the kinetic energy would be confined primarily to the neighborhood of the mouth (where the motion would not be uniform). (The mathematical description of the three-dimensional problem, as it arises in acoustics, is rather simpler because the normalized kinetic energy depends only on the dimensions of the aperture and is independent of the dimensions and shape of the vessel.)

One is led to expect that for nearly closed harbors the second mode (corresponding to $\lambda_0 \rightarrow 2$ d) is more effectively excited than the fundamental mode. This expectation is born out by an examination of the power amplification. For the case b/d = 0.2

	a/b =	.001	.01	.1
fundamental	Q	= 14.0	11.5	8.4
second mode	Q	= 49.7	39.7	15.3

so that the second mode is more highly amplified than the fundamental mode. Moreover, the velocity in the mouth would be relatively much larger for the fundamental mode, so that turbulent dissipation there would decrease the amplification of the fundamental relative to that of the second mode.

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[8] Ibid., p. 303.

Boundary-Value Problem.—Consider a general formulation of the boundary-value problem for the harbor of Fig. 4, assuming a priori only that the disturbance is harmonic and that the barrier is plane.

Let $\zeta_i(x, y)$ be the incident (from $x > 0$) disturbance; for example, $\zeta_i(x, y) = A \exp[ik(x \cos\theta_i + y \sin\theta_i)]$ for a straight-crested wave at an angle of incidence θ_i. If there were no opening in the barrier, the reflected disturbance would be simply $\zeta_i(-x, y)$, and the normal velocity (proportional to ζ_x) would vanish identically at $x = 0$. In fact, the normal velocity differs from zero in the mouth, and the radiated disturbance of Eq. 15 must be superimposed on the incident and (totally) reflected disturbances. Introducing

$$\zeta_x(0, y) = f(y) \quad \text{(29)}$$

the total disturbance outside the harbor then may be posed in the form

$$\zeta(x, y) = \zeta_i(x, y) + \zeta_i(-x, y) + \frac{1}{2}\, i \int_M H_0^{(2)}(kR)\, f(\eta)\, d\eta,\ x > 0 \quad \text{(30)}$$

Turning to the disturbance inside the harbor, let the Green's function $G(x, y, \eta)$ satisfy the Helmholtz equation (Eq. 13) in S and have a vanishing normal derivative on the boundary of S except at $x = 0$ and $y = \eta$, in which $\delta G/\delta x = \delta(y - \eta)$, the Dirac delta function. Then G is real for a harbor of finite area in consequence of our neglect of internal dissipation. Having G, the disturbance inside the harbor may be expressed in terms of $f(y)$ according to

$$\zeta(x, y) = \int_M G(x, y, \eta)\, f(\eta)\, d\eta,\ (x, y) \text{ in S} \quad \text{(31)}$$

Then, requiring the displacement to be continuous across the mouth of the harbor, Eqs. 30 and 31 may be equated at $x = 0$ to obtain the integral equation

$$\int_M K(y, \eta)\, f(\eta)\, d\eta = 2\, \zeta_i(0, y),\ y \text{ in M} \quad \text{(32)}$$

in which

$$K(y, \eta) = G(0, y, \eta) - \frac{1}{2}\, i\, H_0^{(2)}(k\, |y - \eta|) \quad \text{(33)}$$

The exact solution of the integral equation (Eq. 32) for any specific configuration appears to be beyond the available powers of analysis, and therefore approximate methods must be resorted. Assume a solution to Eq. 32 in the form

$$f(y) = F\, \phi(y) \quad \text{(34)}$$

in which $\phi(y)$ specifies the distribution (or shape) of $f(y)$ and F its amplitude. It is also found expedient to normalize $\phi(y)$ according to

$$\int_M \zeta_i(0, y)\, \phi(y)\, dy = \zeta_i(0, 0) \quad \text{(35)}$$

Then, substituting Eq. 34 into Eq. 32 multiplying both sides of the result by $\phi(y)$, integrating over M, and invoking Eq. 35

$$F = 2\, \frac{\zeta_i(0, 0)}{D(k)} \quad \text{(36)}$$

and

$$D(k) = \int_M \int_M K(y, \eta)\, \phi(y)\, \phi(\eta)\, dy\, d\eta \quad \text{(37)}$$

Substituting Eq. 34 and Eq. 36 into Eq. 31, the disturbance in the harbor is obtained in the form

$$\frac{\zeta\,(x,\,y)}{\zeta_i\,(0,\,0)} = \frac{2}{D(k)} \int_M G(x,\,y,\,\eta)\;\phi(\eta)\;d\,\eta \;\ldots\ldots\ldots \quad (38)$$

The preceding formulation is essentially similar to that developed elsewhere[9] for the closely related problem of acoustic radiation from a flanged pipe. A systematic procedure for the determination of $\phi(y)$, based on a variational principle due to Schwinger, has been developed there. Only the following basic principle is stated herein: $D(k)$ is stationary with respect to first-order variations of $\phi(y)$ about the true solution to the integral equation (Eq. 32) provided that the approximate form of $\phi(y)$ is normalized according to Eq. 35.

The resonances of the harbor evidently are associated with the zeros of $D(k)$. These zeros are complex in consequence of the radiation damping, but if this damping is small the resonant frequencies as the zeros of the real part of $D(k)$ may be obtained and also Q by expanding $D(k)$ in a Taylor series about the latter zeros.

Rectangular Harbor.—Now apply the general formulation of the preceding section to the rectangular harbor of Fig. 6 on the assumptions that

$$k\,a \leqslant k\,b \ll 1 \;\ldots\ldots\ldots\ldots\ldots\ldots \quad (39a)$$

and

$$d > b \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (39b)$$

Assume the incident wave to have the form

$$\zeta_i\,(x,\,y) = A\,e^{ik\left(x\,\cos\,\theta_i + y\,\sin\,\theta_i\right)} \;\ldots\ldots\ldots\ldots \quad (40)$$

so that

$$\zeta_i\,(0,\,y) = A\left(1 + i\,k\,y\,\sin\,\theta_i + \ldots\right) \;\ldots\ldots\ldots\ldots \quad (41)$$

in the mouth of the harbor. Substituting Eq. 41 into Eq. 35 and invoking symmetry, the normalization condition on $\phi(y)$ may be reduced to

$$\int_{-\frac{1}{2}a}^{\frac{1}{2}a} \phi(y)\;dy = 1 + O\left(k^2\,a^2\right) \ldots\ldots\ldots\ldots\ldots \quad (42)$$

It is concluded that, subject to the restriction $k\,a \ll 1$, the response of the harbor is independent of the angle of incidence. Also note that, to this same approximation,

$$H_o^{(2)}\;(k\;|\,y - \eta\,|\,) = 1 - \frac{2\,i}{\pi}\,\ln\left(\frac{1}{2}\,\gamma\;k\;|\,y - \eta\,|\right) \ldots\ldots \quad (43)$$

in which $\ln\gamma$ is Euler's constant ($\gamma = 1.78\ldots$)

The boundary conditions to be satisfied by the Green's function for the rectangular harbor are

$$G_x\Big|_{x=0} = \delta\,(y - \eta) \;\ldots\ldots\ldots\ldots\ldots\ldots \quad (44a)$$

$$G_x\Big|_{x=-d} = 0 \;\ldots\ldots\ldots\ldots\ldots\ldots \quad (44b)$$

and

$$G_y\Big|_{y=\pm\frac{1}{2}b} = 0 \;\ldots\ldots\ldots\ldots\ldots\ldots \quad (44c)$$

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9 "The Coupling of a Cylindrical Tube to a Half-Infinite Space," by J. W. Miles, Journal, Acoustical Soc. of America, Vol. 20, 1948, pp. 652-664.

An elementary solution to the Helmholtz Eq. 13 that satisfies Eqs. 44(a) and 44(b) is $\cosh\left[\alpha_n (x+d)\right] \cos(\beta_n y)$, in which

$$\alpha_n = \left(\beta_n^2 - k^2\right)^{\frac{1}{2}} \quad \dots\dots\dots\dots\dots\dots (45a)$$

and

$$\beta_n = 2\, n\, \frac{\pi}{b} \quad \dots\dots\dots\dots\dots\dots (45b)$$

Superimposing these elementary solutions and determining the coefficients of the resulting Fourier series at x = 0 in such a way as to satisfy Eq. 44(a), Eq. 46 is obtained (note that $\alpha_o = i\, k$)

$$G(x, y, \eta) = -\frac{\cos[k(x+d)]}{k\, b \sin(k\, d)} + \frac{2}{b} \sum_{n=1}^{\infty} \frac{\cosh\left[\alpha_n (x+d)\right]}{\alpha_n \sinh\, \alpha_n d} \cos(\beta_n y)\cos(\beta_n \eta) \dots (46)$$

Setting x = 0 in Eq. 46 and invoking the approximations (for $n \geqslant 1$) $\alpha_n \doteq \beta_n$ and $\coth(\alpha_n d) = 1$ by virtue of the restrictions Eq. 39(a) and Eq. 39(b)

$$G(0, y, \eta) = -(k\, b)^{-1} \cot(k\, d) + \sum_{n=1}^{\infty} (\pi\, n)^{-1} \cos(\beta_n y) \cos(\beta_n \eta) \dots (47)$$

Combining Eq. 43 and Eq. 47 in Eq. 33,

$$K(y, \eta) = -\frac{1}{2}\, i - (k\, b)^{-1} \cot(k\, d) - \frac{1}{\pi} \ln\left(\frac{1}{2}\, \gamma\, k\, | y - \eta |\right) + \Sigma(y, \eta) \dots (48)$$

in which $\Sigma(y, \eta)$ stands for the infinite series in Eq. 47. This series may be summed in the form

$$\Sigma(y, \eta) = -(2\, \pi)^{-1} \ln\left[2 \left| \cos\left(\frac{2\, \pi\, y}{b}\right) - \cos\left(\frac{2\, \pi\, \eta}{b}\right)\right|\right] \dots (49)$$

Substituting Eq. 48 into Eq. 37 and invoking Eq. 42, the result may be placed in the form

$$D(k) = -\frac{1}{2}\, i - (k\, b)^{-1} \cot(k\, d) + E(k) \quad \dots\dots\dots\dots (50a)$$

in which

$$E(k) = \int_{-\frac{1}{2}a}^{\frac{1}{2}a} \int_{-\frac{1}{2}a}^{\frac{1}{2}a} \left[\sum(y, \eta) \quad \dots\dots\dots\dots\dots (50b)\right.$$

$$\left. - \frac{1}{\pi} \ln\left(\frac{1}{2}\, \gamma\, k\, | y - \eta |\right)\right] \phi(y)\, \phi(\eta)\, d\, y\, d\, y \quad \dots\dots\dots (51a)$$

and

$$E'(k) = -\frac{1}{\pi\, k} \quad \dots\dots\dots\dots\dots\dots (51b)$$

Eq. 51(b) and hence the dependence of D on k, is independent of the unknown distribution $\phi(y)$; moreover, by virtue of the aforementioned variational principle, the exact value of E(k) is an absolute minimum with respect to arbitrary variations of $\phi(y)$ about its true form. (This last conclusion depends also on the positive-definite character of the integral in Eq. 51(a).)

To compute E(k), the velocity-distribution function $\phi(y)$ must first be estimated. Following, in essence, arguments advanced by Rayleigh[10] in his

10 "Theory of Sound," by Lord Rayleigh, Dover Publications, New York, 1945, p. 307.

treatment of Helmholtz resonators, two approximations naturally suggest themselves, namely

$$\phi^{(1)} = \frac{1}{a} \quad \text{.................... (52a)}$$

and

$$\phi^{(2)} = \pi^{-1}\left[\left(\frac{a}{2}\right)^2 - y^2\right]^{-1/2} \quad \text{.............. (52b)}$$

The distribution $\phi^{(1)}$ would be realized if a rigid, massless piston were fitted in the mouth, whereas $\phi^{(2)}$ would be realized for a uniform flow through an aperture in an infinite barrier $(a/b \rightarrow 0)$. Substituting Eqs. 52(a),(b) into 51(a) and carrying out the integrations

$$\pi\ E^{(1)}\ (k) = \frac{3}{2} + \ln\left(\frac{2}{\gamma}\ k\ a\right) + \left(\frac{b}{\pi\ a}\right)^2 \sum_{n=1}^{\infty} n^{-3} \sin^2\left(\frac{n\ \pi\ a}{b}\right) \text{...(53a)}$$

and

$$\pi\ E^{(2)}\ (k) = \ln\left(\frac{8}{\gamma}\ k\ a\right) + \sum_{n=1}^{\infty} n^{-1}\ J_o^{\ 2}\left(\frac{n\ \pi\ a}{b}\right) \quad \text{...... (53b)}$$

in which J_o is a Bessel function of zero order. Using the approximation $\Sigma(y, \eta) \rightarrow -(1/2\ \pi) \ln\left[(2\ \pi/b)^2 \mid y^2 - \eta^2 \mid\right]$ for small values of y and η, it is also found that $\pi\ E^{(1)} \rightarrow 3 - \ln(b/\gamma\ \pi\ k\ a^2)$ and $\pi\ E^{(2)} \rightarrow \ln(16\ b/\gamma\ \pi\ k\ a^2)$ as $a/b \rightarrow 0$. The difference $E^{(1)} - E^{(2)}$ is 0.001 for a = b and 0.072 for $a/b \rightarrow 0$. Invoking the variational principle, it is concluded that $E^{(2)}$ is the superior approximation; in fact, $E^{(2)}$ is exact as $a/b \rightarrow 0$, for then $\phi^{(2)}$ must tend to Eq. 52(b). The small (numerical) differences between $E^{(1)}$ and $E^{(2)}$, despite the crudity of the approximation Eq. 52(a) attest to the power of the variational principle and imply that neither $E^{(1)}$ nor $E^{(2)}$ can be wide of the mark. The infinite series in both Eq. 53(a) and Eq. 53(b) converge rather slowly, however, and therefore the following approximation is used as

$$\pi\ E(k) = \ln\left[\left(\frac{8}{\gamma}\ k\ a\right) \csc\left(\frac{\pi\ a}{2\ b}\right)\right] \text{............ (54)}$$

which agrees with $E^{(2)}$ as $a/b \rightarrow 0$ and differs from it by 0.036 at a = b. This result, which was obtained from a consideration of the flow in a channel of width b obstructed by a symmetric barrier having an aperture of width a, may be applied to the asymmetric configuration of Fig. 7 if $\csc(\pi\ a/2\ b)$ in Eq. 54) is replaced by $\csc^2(\pi\ a/2\ b)$.

The resonant frequency and Q of the harbor are now computed on the basis of the approximation Eq. 54. Referring to Eq. 4 and Eq. 38 let k_o be zero of the real part of D(k) and expand D(k) about $k = k_o$ in the form given in Eq. 55. (It may be shown, by explicit computation, that the variation of the integral in Eq. 38 with k as $k \rightarrow k_o$ is small compared with that of D(k) for $Q >> 1$.)

$$D(k) = -\frac{1}{2}\ i + Q\left(1 - \frac{k}{k_o}\right) + \text{.............. (55)}$$

Comparing Eq. 50 and Eq. 55

$$\cot\left(k_o\ d\right) = k_o\ b\ E\left(k_o\right) = \frac{k_o\ b}{\pi}\ \log\left[\left(\frac{8}{\gamma\ k_o\ a}\ \csc\ \frac{\pi\ a}{2\ b}\right)\right] \ldots . \quad (56)$$

and

$$Q = \left| k\ \frac{dD}{dk} \right|_{k = k_o} = \frac{\cot\ (k_o\ d)}{k_o\ b} + \frac{d}{b}\ \csc^2\ (k_o\ d) - \frac{1}{\pi} \ldots\ldots \quad (57)$$

Eqs. 56 and 57 are equivalent to Eqs. 24 and 25. Invoking the assumption $Q >> 1$, the variation of ξ with k in the neighborhood of $k = k_o$ depends primarily on D (Eq. 38), then

$$\frac{A_o^2}{A^2} \doteq \left| \frac{D(k)}{D(k_o)} \right|^2 \doteq 1 + 4\ Q^2 \left(1 - \frac{k^2}{k_o}\right) \ldots\ldots\ldots \quad (58)$$

which is identical in form with Eq. 4 if it is recalled that $k/k_o = \omega/\omega_o = f/f_o$.

APPENDIX.—NOTATION

A = amplification factor;

a = entrance width of harbor (Fig. 6);

b = width of rectangular harbor (Fig. 6);

c = phase velocity;

d = length of rectangular harbor (Fig. 6);

E = energy;

f = frequency;

G = Green's function;

g = gravity;

H = Hankel function;

h = depth of harbor;

i = $\sqrt{-1}$;

k = $2\ \pi/\lambda$, wave number;

M = mouth of harbor;

P = flux of energy through harbor mouth;

Q = sharpness of resonance;

$\underset{\sim}{q}$ = particle vector velocity;

R	= radial distance from harbor mouth;
r	= radial distance from harbor mouth (Fig. 4);
Rl	= real part of;
S	= area of harbor;
S(f)	= power spectrum;
t	= time;
u	= orbital velocity;
x	= coordinates (Fig. 5);
y	= coordinates (Fig. 5);
Z	= impedance;
z	= elevation above mean level;
α	= wave numbers;
β	= wave numbers;
γ	= ln γ is Euler's constant;
∇	= differential operator;
δ	= dirac delta function;
ζ	= elevation of surface above mean sea level;
θ	= direction (Fig. 4);
λ	= incident wave length;
ρ	= density; and
ω	= circular frequency.

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DISCUSSION

MEAN DIRECTION OF WAVES AND WAVE ENERGY[a]

Closure by Omar J. Lillevang

OMAR J. LILLEVANG,[6] M. ASCE.—Dunham related the alignment and history of natural shoal forms and raises the point that shoals resulting from the influence of artificial structures may eventually develop similarities to the natural. Sandy Hook, at the north end of the New Jersey Coast, may well be nature's analogy of the breakwater tip shoal, particularly at a location at which the energy flux of major river flows is an element compounding the complications of wave action. Sandy Hook's recurved end may exist to a considerable extent because of the river effect and also due to the local chop of waves generated in Lower New York Bay by offshore winds. Neither complicating effect is a possibility at the locations discussed in the paper.

The Santa Barbara record appears to support Dunham's suggestion that as inshore contours move eastward in an accretion area the outline of deposits, or the alignment of shoal deposits, is forced seaward. Fig. 6 and the preceding remarks concerning it, bear on this concept. The bottom contours paralleling the Santa Barbara breakwater did move seaward quite consistently after the structure was built, about 1930 to 32. Whatever trend lines one might elect to draw through the individual survey data plotted on Fig. 6 would show a swing to the seaward of the shoal limits. It is planned that expected future shoal developments at Del Mar Harbor will be removed frequently to maintain navigation, as they have been at Santa Barbara. Thus, except for the different wave exposure, the Oceanside project may be comparable to the Santa Barbara one.

Bruun correctly takes the writer to task for referring to energy as a directed phenomenon. Here simplification may have been overdone to avoid a title for the paper that might otherwise have read "Mean Direction of Waves and of the Flux of their Energy at Coasts and Barriers."

It is encouraging that the problems discussed in the paper have brought out discussions from the heads of two of the world's exceptional coastal engineering laboratories. Their independent references to model study for development of sound theory is impressive. The hope exists that such work may be recognized as a very useful thing for allocation of research funds. The successful outcome of such research would be a reduction of the empirical and an increase in the rational practice of coastal engineering.

Jordaan's interest in the uniform longshore current is not intensely shared by the writer. However, probably this is because none of his experience has encountered such a beach-paralleling current swift enough to move bed load, as would be done by a stream or by swift tidal flow at an estuary. On many occasions along the California coast when waves were breaking obliquely to a

[a] March, 1960, by Omar J. Lillevang (Proc. Paper 2423).

[6] Vice Pres., Leeds, Hill and Jewett, Inc., Cons. Engrs., Los Angeles, Calif.

long, straight shore, the writer has noted pronounced longshore currents in the surf zone that were set in the opposite direction one might expect from the oblique incidence of the waves.

Jordaan has suggested that "the Q-factor equation might be improved by utilizing w $\sin^2$ I rather than w Sin I Cos I." As derived by the Los Angeles District of the Army Engineers, the latter resulted from the following reasoning:

1. Energy content per unit length of wave crest offshore is representable as w;
2. At the shoreline the energy content per unit length of wave crest has been modified by refraction, diffraction, shoaling depths, island screening, and so forth and is, thus, represented by w E;
3. The wave crests at shore may not be parallel to shore. Thus, the energy content per unit length of shoreline, if I is the angle between crest and shore, is computed as w E Cos I;
4. The flux of the above unit energy per foot of shoreline has an incidence angle at the littoral zone, I: Thus, the component of that flux longshore is the product of the energy flux and the sine of the incidence angle, or w E Cos I Sin I, that was confusingly written in the paper as w E Sin I Cos I, and properly reduced to $\frac{1}{2}$ w E Sin (2 I).

The writer closes this discussion with sincere appreciation for the criticisms and additions that Dunham, Bruun and Jordaan have contributed and with an expression of hope that correspondence or presentations in the literature of their contributions may follow.

DESIGN CONSIDERATIONS FOR CALIFORNIA MARINAS[a]

Discussion by Omar J. Lillevang

OMAR J. LILLEVANG,[21] M. ASCE.—The author has emphasized that the prediction of direction and magnitude of waves by rational refraction and diffraction computations can be made and that harbor designers should learn and use these procedures, or commission those already skilled to perform them. Another wave-induced phenomenon is more difficult to predict mathematically, unless the harbor channels and basins approximate simple geometrical forms separated by sharp constrictions or other simplifying limits. The problem referred to is resonance. At one California small craft harbor, waves of certain critical frequencies, which come in the entrance channel from the ocean, pass a constricted side entrance to a large mooring basin. They induce a resonating surge in the basin which has torn fittings from boats, broken mooring hardware on slips, severed lines and otherwise made the basin unattractive to owners of boats who would willingly rent moorings there. With hindsight, it is clear that rather simple model studies of the harbor in the design phase might have avoided what may now prove expensive, and certainly will be inconvenient remedial measures. Not the least expense, by any means, is the bad reputation the surges have made for the harbor among boat owners.

At most harbors on an open coast, wave approaches from many directions are to be expected, because the waves are propagated in storm centers which may occur anywhere in the oceans. Thus, in general, there is no alignment of an entrance which will not at some time have waves moving to the inner areas with only slight attenuation of their offshore characteristics. It follows that some type of energy absorptive works at the harbor end of the entrance is desirable. A gently sloping beach directly across the direction of the channel is superior to other devices, but often cannot be provided because of land limitations or location of continuing channels to the inner harbor areas. Vertical barriers should be avoided at all costs, because they almost totally reflect the waves back on themselves and the result is a series of standing waves in the vicinity which have twice the amplitude of the waves prior to reflection. Care must be taken with sloped boundaries, when they cannot be as gentle as a beach, that they be built of rough, porous rubble or otherwise be absorptive, lest they reflect the incident waves with results nearly as drastic as those of the vertical barrier.

Providing maneuvering room for sailing craft without auxiliaries can be an expensive luxury. The number of such boats is diminishing rapidly and the limited number of slips needed to provide for these severe space-consuming considerations should be located along main channels or basins. Likewise, to

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[a] November, 1960, by James W. Dunham (Proc. Paper 2658).

[21] Vice Pres., Leeds, Hill, and Jewett, Inc., Cons. Engrs., Los Angeles, Calif.

adopt a lenght-to-width ratio of slips to allow for the occasional "square" hull imposes a cost in terms of water area consumed which is unjustified. It seems reasonable to expect the owner of the abnormally wide boat to meet the greater rental expense of a longer slip and thus have the required width.

Fig. 2 is particularly interesting when compared with the record of nearly 9,000 boats owned in Orange County, the area in which the famed Newport Bay is located in southern California. In planning for two new harbors, each to provide for more than 500 boats in initial development, the writer analyzed punch card data for every boat more than 15 ft long registered in Orange County in 1960. Fig. 14 presents those data by percentage distribution of lengths, and the San Francisco Bay Area data are drawn as a dotted line for comparison.

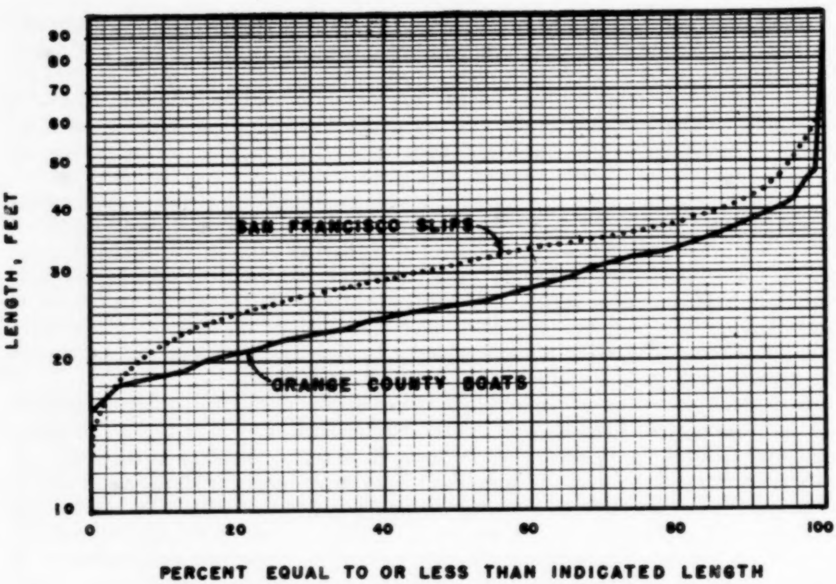

FIG. 14.—PLEASURE CRAFT REGISTERED IN ORANGE COUNTY IN 1960

Apparently a marina designed for southern California size distributions would not fit the demand in San Francisco Bay.

Analysis of the same punch card record for Orange County boat ownership reveals that virtually all boats more than 27 ft long are moored in marinas the year around, and practically none less than 19 ft are kept in slips at all, but are trailered from dry storage to the water. Marina operators report that the expense of managing slip rentals for boats less than 25 ft long is disproportionately high because of the readiness of their owners to terminate rental agreements and remove their boats to dry storage. The situation is perhaps comparable to the difficulties of managing small furnished apartments where occupants are highly transient. A difference lies in the insistence by owners

of the small boats that they pay no more rent per foot of slip than do the owners of larger boats. One compensating factor is that the cost of land, whether already water-covered or dredged to make installation of mooring feasible, is porportionately greater for the larger slips than for the smaller, because broader expanses of maneuvering areas per slip are necessary for the large boats. Whatever distribution of slip sizes one may adopt after studying the local statistics, the majority of boats will be in slips longer than the individual needs, if the usual rule is enforced that no boat may extend into the approach channel. Unless a large marina is built "from scratch," it is not economical to break the slip sizes down to as small as 5 ft increments of length. A current design for 575 slips in a southern California harbor is distributed to fit the Orange County record as follows:

Length of Slip, in feet	Number	Percentage Equal or Less than Length Tabulated
25	250	44
33	205	79
43	90	95
51	20	98
More than 51	10	100

In a marina, the floating facilities are more or less open to the walking public, which includes the unsteady on foot who are young, or old, or wearing spike heels, or full of "good cheer." Perhaps none of these should be on the floats unassisted, but they are. Thus the stability of the floats is important and quickness of response by floats to wave action, or boat impact, or any other moving load, deserves careful consideration. Also, working of joints in highly flexible structures often becomes a maintenance problem. Flexibility of deck systems and light weight, small displacement flotation elements should be avoided to the extent that cost and connection stresses will allow. With framed decks bridging from pontoon to pontoon, it is reasonably easy to design for stability against horizontally applied loads on the slip fingers and walks. The deck system can be cross-braced and act as a deep truss laid on its side. However, it is difficult to design a system so shallow, vertically, to resist torsional displacement with eccentric vertical loading. Assemblies of concrete boxes, tied together with plank facias and with their upper surface serving as the walking deck, have been used for slips and develop great assistance to torsional displacement. This is principally because their monolithic performance as a deep girder resists distortion under any eccentric loading which would not otherwise sink the assembly.

Recently the writer went to their established marinas, two of which the slips were wood frames on lightweight floats and the third was a continuous concrete float system. The more reactive wood frame fingers, 40 in. wide, deflected 2 1/8 in. under a load of 165 lb, applied vertically 6 in. in from one edge. The stiffer wood frame, 44 in. wide, deflected only 3/4 in. under the same load, but for an all-concrete float system slip finger only 34 in. wide, it required 280 lb 4 in. in from one edge to develop a 1/8 in. deflection. The relative stability was even more dramatic under quick load, applied as a rapid shifting of weight from one side to the other or by jumping from the decks of boats in the slips to the walking surface of the floating slip enclosures.

LATEST DREDGING PRACTICE[a]

Discussion by Charles E. Behlke, John B. Herbich, and Alf. H. Sorensen

CHARLES E. BEHLKE,[2] M. ASCE.—The writer would like to offer a few statistics and practices of dredging in The Netherlands that are interesting to compare with those of American dredging given by the author. Most of the information provided was given to the writer by Ir. H. T. den Breejen.

Some of the most striking differences between Dutch and American dredging practices are illustrated by the following figures on Dutch dredges for the year 1957.

Bucket Dredges	265
Suction Dredges	204
Hopper Dredges	18
Dripper Dredges	1
Barges to haul dredged material	1231

The Dutch use a relatively large number of bucket dredges. This is in sharp contrast with American practice where a bucket dredge is seldom used except, perhaps, in mining operations. Suction dredges in Holland are sharply on the increase. The figures also indicate the relatively large number of barges used.

Generally, Dutch dredges are smaller than those in America and the crews are frequently housed on the dredges of any size.

Dutch bucket dredges are usually rated by the size of the buckets. Hence, on a 400 l dredge, each bucket would have a capaicty of 400 l. On the average, Dutch bucket dredges cost approximately 3,000 Dutch Guilders per l of capacity (abbreviated f. 3,000) (Approximately f. 3.7 = \$1.00). On the basis of f. 3,000 per l per bucket, the following figures would apply approximately to a 400 l bucket dredge.

Initial cost of dredge = 400 (3,000) = f. 1,200,000

1.	Maintenance including dry dockage at 8% of initial cost per yr	= f.	96,000
2.	Insurance at 3% per yr	=	36,000
3.	Sinking fund at 6% interest and 25 yr life	=	21,900
4.	Interest on initial cost at 6%	=	72,000
5.	Overhead at 3% of initial cost	=	36,000
	Total fixed cost per yr	= f.	261,000

[a] February 1961, by Ole P. Erickson (Proc. Paper 2729).

[2] Assoc. Prof. of Civ. Engrg., Oregon State Unov., Corvallis, Oreg.

Operating costs per week of operation

1. Wages		= f.	1,000
2. Social insurance			500
3. Coal			2,000
or diesel	f. 500 per week		
4. Store sundries			800
Total weekly operating costs			
for a coal operated dredge		= f.	4,300
for a diesel dredge		= f.	2,800

The average Dutch dredge works approximately 30 weeks per yr, so the weekly charge of operation is the direct operating cost plus the fixed costs prorated more than 30 months. This amounts to f. 4,300 + (f. 261,000/30)

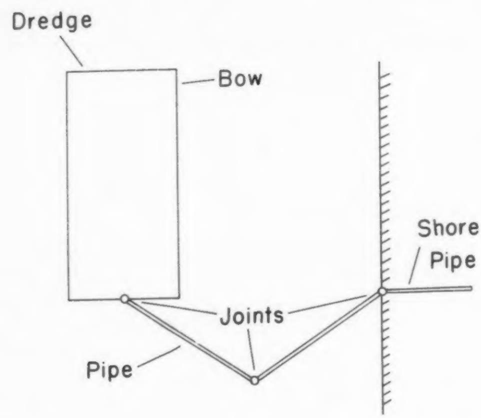

FIG. 10.—THREE JOINT PIPE CONNECTING METHOD

= f. 13,000 per week of operation. Of this, manpower costs only f. 1,500, or approximately 11.5%. The best manpower can be sought because there is little difference in cost between good and poor crews. However, earnings are almost directly a function of how well the dredge is operated. The relatively small cost of labor also explains the fact that Dutch dredges are usually less automated than American dredges.

Some of the Dutch methods of operation are of considerable interest. The writer observed that no anchors were handled by booms on Dutch dredging operations. Auxiliary boats usually lift and place the mooring anchors.

Many Dutch dredging firms like to use the three joint method of connecting their pipe between the shore pipeline and the dredge. This is shown in Fig. 10. This method only works well in calm water, but it allows the dredge to cover a great area without handling any piping.

Occasionally, the Dutch use cylindrical pontoons to support the floating pipeline. These cylinders are concentric with the supported pipe as shown in

Fig. 11. This type of pontoon works especially well in rough weather, but it has a glaring disadvantage because leaks in the pontoon are quite difficult to find.

Another type of pontoon is shown schematically in Fig. 12. This type of pontoon has a removable top on each segment allowing several to be stacked like pans when the dredge is moved.

Few of the Dutch dredges are self-propelled. They feel that, because during operation the dredge is only moved a few times a day, it is cheaper to have a small tug provide the necessary propulsion, thus saving space and capital investment.

Some of the bucket dredges have hoppers into which the dredged material is dumped and from which it is picked up by a suction line and put into the discharge pipe. This is essentially a dredge within a dredge. It would seem better to make the dredge a suction dredge to begin with, but the Dutch feel that for many materials such as clay, the bucket dredge works more efficiently than a suction or suction cutter dredge.

The writer has recently received information that IHC Holland, a combine of six Dutch companies that builds and repairs ships and dredges, is presently constructing two cutter suction dredges of the stationary type, each having

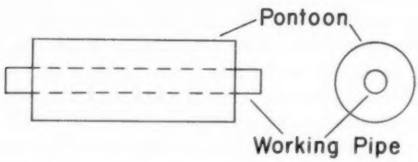

FIG. 11.—CIRCULAR PONTOON

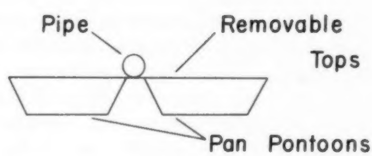

FIG. 12.—PAN TYPE PONTOON WITH REMOVABLE TOPS

5,300 hp. They will be the largest dredges ever constructed by these Dutch contractors. The dimensions of these dredges are 190 ft by 44 ft by 14 ft.

It is also interesting to note that IHC Holland operates a testing laboratory in Delft that seeks improvements in dredging methods. Here models are constructed and tested under controlled conditions. Much valuable information has been obtained in this laboratory. The writer knows of no other laboratory of this type in the world.

The Dutch construct many dredges and own and operate so many that during the depression of the 1930's, there was not enought room in home harbors to moor all of their unused dredges that were working throughout the world and at home at the beginning of the depression.

While in Holland, the writer witnessed the aftermath of a serious dike break at Tuindorp-Oostzaan in January, 1960. Immediately following the dike break, five large suction dredges that were in the vicinity were moved to the afflicted area. As soon as the dike repair was affected, the dredges placed their suction lines in the flooded area and proceeded to pump out the flood water, dredging only water and no soil.

The new subway for the City of Rotterdam will pass through the center of the business district. The Dutch will not tunnel to accomplish this construction, but will work in a large open trench that will be created by dredging.

When the Maas tunnel was constructed under the Maas River in Rotterdam, the construction was performed without tunnel. A trench was dredged across the river and precast tunnel sections 200 mi long were floated into position and sunk in place. These sections were then connected in a watertight manner and the water removed, thus creating a tunnel with dredges performing the necessary excavation.

The preceding examples illustrate the fact that the Dutch, besides being a trading nation, are also a dredging nation. The writer does not wish to imply that Dutch methods are better or worse than American methods. The two cannot be compared because the financial structure in the two countries with regard to labor is so completely different. This information has been offered to indicate some of the practices in another country that are quite important in the world of dredging.

JOHN B. HERBICH,[3] M. ASCE.—The principal intention of the writer is to supplement the paper and to briefly summarize the current research program at Lehigh University aimed at improving the efficiency of dredge pumps, particularly for pumping silt-clay-water mixtures.

In analyzing the history of hydraulic dredging (the principle of dredging by means of a centrifugal pump), mention should be made of the existence of a hydraulic hopper dredge General Moultrie[4] in the United States in 1855. The dredge pump with an impeller of approximately 6-ft diam, was revolving on a vertical axis, its 19-in. diam suction pipe with a bell-mouth lower end resting on the channel bottom, while it discharged the dredged material directly into a "hopper" in the vessel. The pump was moved by the steam engine which was also used to propel the ship. The records indicate that on the average, 328 cu yd of material was dredged per working day.

It appears, therefore, that the hydraulic or suction principle was first used for dredging in the United States. The General Moultrie became a casualty of the Civil War, and dredging by hydraulic means was not tried again in the United States until 1871, when a steamer, Henry Burden, was converted for suction dredging and used in improving the mouth of the St. John River, Fla. The dredging equipment of the Burden consisted of a 9-in. centrifugal pump, a 6-in. suction pipe on each side, tee-connected to the single pump, and two 6-in. pipes tee-connected to the 9-in. pump discharge.

A great number of hopper dredges were either purchased or built by the Corps of Engineers between 1891 and the presnet time, culminating with construction of hopper dredge Essayons in 1949. The Essayons is 525 ft long, has a hopper capacity of 8,000 cu yd, maximum dredging depth of 60 ft, and is equipped with two 36-in. suciton - 32-in. discharge dredge pumps, 1,850 hp each. Construction of new hopper dredges by the Corps of Engineers since 1936 was aimed to replace the older dredges. Because of the improved efficiency of the modern dredges with their larger hopper capacity, greater speed, and better maneuverability, the number of dredges operated by the Government

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3 Assoc. Prof., Chrmn., Hydr. Div., Fritz Engrg. Lab., Civ. Engrg. Dept., Lehigh Univ., Bethlehem, Pa.

4 Journal, Franklin Inst., Vol. 32, 3rd Series, No. 6, December, 1856.

has decreased. The complete history, development, and operation of the Corps of Engineers dredges are described elsewhere.[5]

In describing various hydraulic dredges, mention might also be made of the "portable" type dredges. These are hydraulic pipeline cutterhead type with main dredge pump driven by a high-speed Diesel engine through a reduction gear. The hull is approximately 52 ft by 20 ft by 4 ft, and they can be disassembled and transported overland to another location. The pump has a 13 1/4 in. suction and 12-in. discharge, and is operated by a 260 hp motor.[6] It has a maximum digging depth of 26 ft and is capable of pumping distances up to 3,000 ft, with outputs varying from 100 to 300 yd per hr in normal materials. The protable dredges which were built for the Indonesian Government may be used to great advantage by local authorities or the contractors.

The trend is away from crew's quarters on dredges, however, the current practice on large dredges is to provide quarters sufficient to permit the dredge to operate on a 24-hr schedule when necessary. Also, the dredges operating in remote areas, such as the two dredges recently built for the Brazilian Government, contain living quarters for forty-five officers and men.

The author mentions that the discharge vane angles at tip vary between 20° and 30°, and entrance angles vary 16° to 24°. The writer finds that the discharge vane angles varied anywhere from 22 1/2° to 35°, and even 67° in the older dredge pump. However, the trend seems to be to reduce the discharge vane angles; for example, the recently built dredge S. S. Zulia in Japan has a discharge angle of 22 1/2°. Such low angle is usually recommended for pumps handling water,[6] and it has not been used on dredge pumps until recently.

The writer finds that the entrance vane angles vary from 37° to 40°; the S. S. Zulia and Essayons having an angle of 45°.

The author mentions a number of empirical formulas for computing friction in pipelines and rightly states that unless reasonably correct allowances are made, the computations may be misleading. There appears to be a great research need to determine the effect of concentration of the solids in water, the grain size, and distribution on the friction factor f. Observations in the laboratory indicate that when the concentration of solids is low (up to 1,200 g/l), the resulting mixture is essentially water with solids in suspension, and the solids settle readily. However, when the concentration is high (up to 1,400 g/l), the mixture appears to be homogeneous. It has properties of a non-Newtonian fluid and the solids do not settle readily. The author mentions that the dredge may pump up to 40% solids. This is misleading unless further clarified whether the percentage solids is by weight or by volume. This, too, may be misleading unless the "solids" are defined. "Solids" as they are sometimes referred to in dredging practice, are actually comprised of the dry voidless grains plus the water which occupies the void spaces between grains. To avoid confusion, these could be called "solids" as "in situ" material, or "bottom material." The density of material expressed in grams per liter with its percentage by volume equivalent will be compared. The total weight may be expressed as

$$x\ (S.G.) + (1000 - x)\ (1) = 1400 \qquad (1)$$

5 "The Hopper Dredge," by F. C. Scheffauer, Editor-in-Chf., U. S. Govt. Printing Office, Washington, D. C., 1954.

6 "Centrifugal and Axial Flow Pumps," by A. J. Stepanoff, John Wiley and Sons, Inc., New York, 1948.

in grams per liter, in which x = cubic centimeter of true solids and S. G. = specific gravity of solids.

Assuming the S. G. of solids = 2.60, x = 250 cu cm of true solids, leaving 750 cu cm of water. Thus the mixture contains 25% by volume of true solids.

A study was initiated by Lehigh University, under the sponsorship of U. S. Army Engineers, Philadelphia District, in 1958, with an object of improving the design of a dredge pump primarily for pumping silt-clay-water mixtures.[7] The affect of impeller design on pump efficiency was studied in some detail, particularly for the silt-clay-water mixtures. The discharge vane angle was varied between 22 1/2° and 35°, as well as the vane shape in the 1:8 scale model pump of Essayons dredge pump. It is not intended to present the model study results here, however, it should be indicated that the pump efficiency can be increased for 71% to 76% for 1,380 g per l, by changing the vane shape from a radial to an involute curve. Also, a change in the discharge vane angle can produce an increase of efficiency from 80% to 82%, and 76% to 83% for the prior-mentioned densities, respectively.

ALF H. SORENSEN,[8] A. M. ASCE.—The author is correct in stating that there is quite a bit of disagreement among dredge designers and builders as to the exact methods and procedures in this field both as to equipment design and its job application. A large majority of dredge people today (1961) do agree on the certain facts and some of these deserve the following comments.

Standard equipment built today by United States and European manufacturers feature 10-in. hydraulic pipeline dredges with hull sizes as small as 40 ft long, 14 ft wide, and 4 ft deep.

A large majority of modern dredge pumps presently built and used by major United States and Canadian contractors are single suction, volute type pumps with a one-piece pump case (or cast in two halves on larger dredges). The engine side head and the suction side head are lined with either Ni-hard or diamond alloy or abrasion resistant steel liners. Usually the pump case and the impeller is made from steel alloy castings from either one of the three general groups of steel such as (a) Abrasion resistant carbon alloy steel, (b) Chrome-nickel-molybdenum alloy steel, heat treated to a high Brinell hardness for abrasion resistant properties, and (c) Manganese steel (manganese steel is used when pumping gravel where the gravel causes impact hardening and increases the abrasion resistant qualities).

The pump heads are made from either cast steel or fabricated steel and do not require abrasion resistant qualities.

Fully lined pumps with fabricated cases are mostly used in applications where extremely abrasive sand and gravels are handled. When abrasion qualities become the principal design criteria, the pump parts which are in contact with slurry mixture is usually made of Ni-hard or of diamond alloy. These alloys, however, have little tensile strength and require an outer fabricated casing to obtain the necessary structural or tensile strength in the pump as a whole.

A check with the major contractors and pump manufacturers in the United States shows that a substantial majority of these disagree with the author in

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7 "Characteristics of a Model Dredge Pump," by J. B. Herbich, Fritz Lab. Report No. 277-PR. 31, Lehigh Univ., 1959.

8 Civ. Engr., Ellieott Machine Corp., Baltimore, Md.

the statement that fully lined pumps are more economical than conventional pumps.

In almost no practical case can a 10-in. dredge pump, pumping through 10-in. I. D. pipeline absorb as much as 600 hp. The majority of 10-in. pumps built today (1961) are designed for power applications between 100 hp and 400 hp. Most manufacturers have standardized on power range between 200 hp and 300 hp.

While the first cost of "built-up cutters" (or cutters welded together from castings or structural steels) is low, the long-term operation economy still

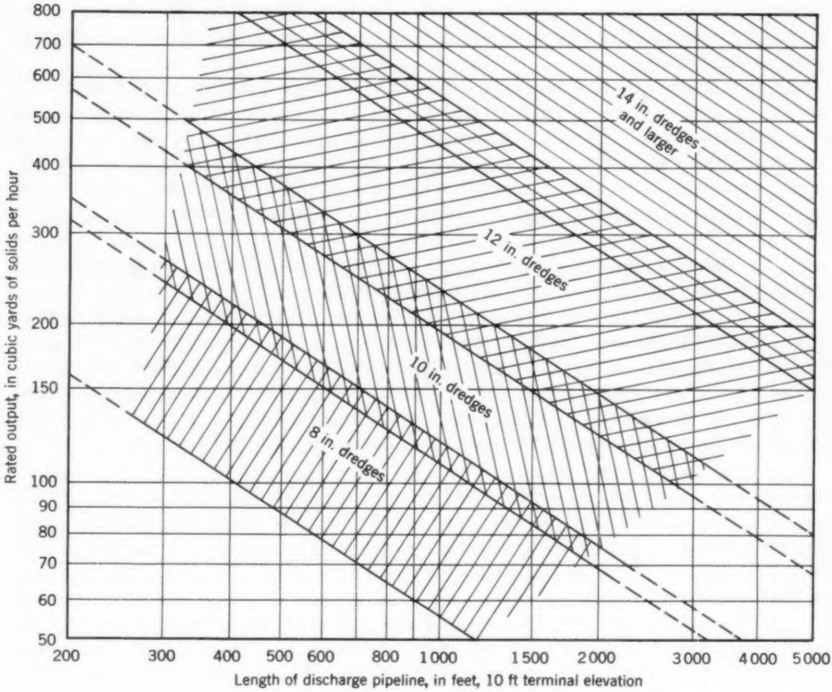

FIG. 1

favors the one-piece casting type cutter or with bolted-on blades. The one-piece cutter is still common.

Many designers recommend a cutter-shaft-thrust bearing to be installed as a separate unit outside and forward of the reduction gear, instead of as an integral part of the reduction gear itself. The reason is that when a bearing failure occurs, particularly on large dredges, it is more economical and time-saving to repair a separate thrust bearing instead of dismantling the entire reduction gear.

The direct suction pipe cutter drive was first used when hydraulic dredges came into common use in the United States in the beginning of the 1900's and

already at this time was shown as an impractical design. The majority of contractors agree that this system is impractical from a maintenance point of view.

It should also be mentioned that recent years have brought into existence hydraulic drives for winches and cutters on dredges.

Mention is made of a 40% solids content in dredge pipeline slurry. It should be emphasized that this is a 40% solids by weight. It is, however, more common to use percentage by volume since dredge material is always mentioned by cubic yards or cubic meters and in this case the 40% figure would correspond with approximately 20% by volume.

It still remains to be proven theoretically as well as empirically that a direct suction pipe cutter drive gives a less water vacuum than a conventional cutter suction design.

The maximum production in a hydraulic dredge system is basically the function of the pipeline velocities, but this is only true where the dredge pump has a positive feed. That is, where the solids are mixed with the water at a predetermined ratio in front and above the suction inlet of the pump. On hydraulic dredges, however, the production is a function of the suction velocity and the ability of the cutter and suction head to feed the suction end of the ladder.

Fig. 1 shows how one manufacturer in the United States qualifies dredge capacities for smaller hydraulic pipeline dredges.

To base the capital cost of the new dredge on the horsepower alone may be quite misleading without qualifying whether it is a diesel, diesel partial electric, diesel electric or all electric dredge. A recent investigation of dredges built in the United States shows that dredges in the sizes 8-in. up to 16-in. vary in price from $180.00 to $350.00 per hp. Larger dredges in the sizes 20-in. through 36-in. vary from $275.00 and as high up as $600.00 per hp.

MARINE OIL TERMINAL FOR RIO DE JANEIRO, BRAZIL[a]

Discussion by Glenn B. Woodruff, Richard S. Winkler, and Joseph H. Finger

GLENN B. WOODRUFF,[10] F. ASCE.—The determination of berthing and mooring forces and the most efficient means of providing plays an important part in the economics of the design of a fuel handling pier. The author has given an excellent example of such a design. In this particular case, the location was such that wind currents and waves were a minor consideration. In the general case, the mooring rather than the berthing forces may be controlling.

Referring to Fig. 3, many designers prefer to set the breasting dolphins ahead of the hose handling platform so that the tanker does not come into contact with the latter. This eliminates any movement of the platform in reference to the submarine lines and reduces the amount of fendering required.

While the location is well-sheltered, the tanker may be subject to winds of 33 knots, currents of 1 knot and waves 5 ft high. Precise data for computing the forces resulting from these causes are not available. Especially with no more flexibility than is provided, the mooring forces against the fenders may well be at least in the same general order as those computed during berthing.

Fig. 6 gives an excellent picture of various conditions during berthing. The reduction coefficient allows for the distance between the center of gravity of the tanker and the point of impact. The author has neglected the hydrodynamic mass that may be considerably greater than the mass of the vessel, the division of the impact energy between the tanker and the structure and wind, wave, and current forces during berthing. This entire matter is complicated; the designer has the option of selecting such approach velocities and angles that will permit of great variation in the potential energy in the fender system. An analysis of the available literature leads to the conclusion that average and presumably satisfactory practice be expressed by

$$E = \Delta (0.004 - \Delta \times 10^{-8} \quad \ldots\ldots\ldots\ldots\ldots \quad (6)$$

in which E is the potential energy transmitted to the fender, in ft tons, and Δ represents the displacement of vessel, in tons.

The assumptions for various designs range from 0.50 times to 1.50 times those given by Eq. 6. The smaller value may be used when winds, currents, and waves are negligible; the larger ones when such conditions are severe. For the author's case of a vessel of 137,000 tons, E becomes 375 ft tons as against the 249 ft tons used.

In detail design of the fender system, the writer prefers to secure greater flexibility than the author proposes. This is especially important, because the

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[a] February, 1961, by H. W. Reeves (Proc. Paper 2733).
[10] Cons. Engr., San Francisco, Calif.

wave forces fall off rapidly with increased flexibility. To achieve such results the writer combines flexible breasting platforms with the fendering. While for this particular case the assumption of 100 ton bollards may be sufficient, the writer suggests that this should not be taken as a precedent by others. The designer has no control over the number and strength of the lines the tankers master will use. The U. S. Navy has adopted as a standard for aircraft carriers that are considerably lighter than the supertankers, bollards of 200 ton capacity.

None of the preceding should be considered a criticism of the author's design but rather as suggestions.

RICHARD S. WINKLER,[11] A. M. ASCE.—The terminal, while small, considering the number of berths available, is of interest in that a number of modern advances have incorporated in its design which give it much operational flexibility. The use of positive displacement flow meters, telemetering, and the successive use of pipe lines for differnet products without causing contamination are all of particular interest. Although the writer is more familiar with the structural aspects of such a terminal, a paper on these operation features would also be greatly appreciated.

In calling attention to the problem of choosing an approach velocity for a berthing ship, the author has emphasized one aspect of structural design subject to the most arbitrary sort of personal opinion. Perhaps this is due to a difficulty in discriminating between a reasonable design condition and an accident. At any rate, as tankers grow larger and as terminals must be placed in more exposed locations, these design problems assume greater importance.

Maximum Tanker Size.—The rate at which the size of the newest tankers has grown since World War II is awesome. No other class of ship has shown such rapid increase in size in so short a period, and it is likely that this rate of growth shall continue. At the present time (1961) two 130,000 DWT tankers are on order in Japan. These ships will cost $14,000,000 and will have a capacity of 900,000 bbls. Their proposed dimensions are: length overall 955 ft, beam 141 ft, depth 73 ft and draft 54 ft. These vessels will be driven by steam turbines rated at 28,000 shp at speeds up to 16 knots. Although the present surplus of shipping tonnage has lead a number of oil companies to suggest that perhaps the limit in the size of tankers has been reached, it can be seen from Table 2 that cost advantages exist for tankers as large as 200,000 DWT and possibly larger. There are a number of technical problems foreseeable in designing and constructing tankers of this size but these problems will undoubtedly be solved.

As seen from Table 2, in comparison with a 50,000 DWT tanker, a 150,000 DWT tanker is expected to reduce transport costs by one third. The increments of savings become proportionately less as size increases, but not to the extent that a 200,000 DWT tanker would appear to be a poor investment compared to a 150,000 tonner if the quantity of oil to be transported is sufficient to insure its full utilization and terminal facilities are available to avoid inordinately long port time. The larger the vessel, the greater the penalty for any idle time, probably the principal reason why larger tankers than 105,000 DWT have not yet been built.

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[11] Structural Engr., Arabian-Amer. Oil Co., The Hague, Netherlands.

It is also interesting to note that for the 200,000 DWT tanker the cost of transporting a barrel of crude oil is 10 to 20 times the direct operating and capitalization cost for a typical terminal. It is thus unlikely that the cost of such a terminal will be any deterrent to the future use of such a tanker. The estimated dimensions for the 150,000 DWT and 200,000 DWT tankers are as follows:

Dimension	150,000 DWT	200,000 DWT
Length overall	1,000 ft - 1,100 ft	1,000 ft - 1,200 ft
Beam	150 ft - 170 ft	170 ft - 180 ft
Draft	51 ft - 55 ft	55 ft - 60 ft

Since the water depth at this new island wharf is 59 ft at mean low water, it should be possible to berth tankers as large as 150,000 DWT and still provide allowance for heave, squat, and some navigable clearance.

Ship Handling Flexibility.—The author is correct in stating that a modern oil terminal need not necessarily be designed as a single continuous pier or

TABLE 2.—ESTIMATED COST OF CARRYING CRUDE OIL FROM THE PERSIAN GULF TO RIO DE JANEIRO, BRAZIL

Ship DWT Classification	50,000	100,000	150,000	200,000
Round Trips per Year	7	7	7	7
Total Fuel Cost per Year (M$)	580	1160	1570	1910
Operating Costs per Year (excluding fuel oil - M$)	840	1000	1160	1290
Interest and Amortization per Year (M$)	980	1420	1860	2300
Total Cost per Year (M$)	2400	3580	4590	5500
Total Crude Oil Capacity per Year (MBbls.)	2500	4900	7200	9700
Transportation Cost ($ per Bbl.)	0.96	0.73	0.64	0.57

dock structure, and that an island wharf with local strong points has probably the lowest possible first cost. On the other hand, however, a continuous face wharf with regularly spaced bollards does have a certain flexibility not obtainable with a design in which separate structures are provided.

Unfortunately there is still a lack of uniformity among tankers regarding fittings, mooring provisions, and cargo loading and discharging facilities. Such lack of uniformity causes considerable expense and difficulty to terminal operators. It would seem a worthy goal for the industry to achieve as much uniformity as possible in tanker design, particularly on the question of cargo unloading and discharge facilities. There is a normal tendency to increase manifold capacity in the larger ships. In the newer vessels the cargo handling system is generally divided into four sections, each with its own line to the manifold location. The main headers and crossovers are usually of 14 in. or 16 in. pipe, however, there may be a wide range in pipe sizes as shown by the five 16-in. lines of the Esso Gettysburg (47,400 DWT) and the four 12-in. lines

of the Universe Leader (85,500 DWT). Tankers built by oil companies and their affiliates usually have a greater capacity in the basic cargo handling facilities as well as more special cargo handling equipment than do those built for charter or speculation. The normal location for the manifold connections is from 45% to 50% of the ships length from the bow. All of the present tankers except those of the Universe Leader class have a single manifold. These have two manifolds located approximately 46% and 57% of the length from the bow. The connections vary from 2 ft-6 in. to 5 ft-0 in. above the deck and from 10 ft to 20 ft back from the ships side. The principle purpose of having the two manifold locations appears to be only to offer more flexibility in making connection to the wharf's pipe lines.

There is also some variation regarding the number and types of mooring lines. The newer ships are equipped with a variety of chocks and winches for wire or manila rope, or both. Power winches are located on the bow and stern and some ships have constant tension winches. The newest Esso tankers carry $1\frac{1}{2}$ in. steel cable with a breaking strength of 50 tons, while the braking power of the winches is 45 tons. The W. Alton Jones has six mooring winches with $1\frac{1}{2}$ in. wire rope, and the 105,000 DWT tanker has ten mooring winches. In the case of a wharf with isolated strong points it is difficult to plan efficient mooring arrangements for the whole range of vessels from coastal tankers of 2,000 DWT to super tankers of 100,000 DWT. If mooring diagrams for the various ships were made based on the known locations of mooring fittings and manifolds, it is likely that the flexibility of a continuous face wharf would be apparent. In addition it is likely that in the case of the super tankers, the mooring dolphins which are opposite the breast lines will be heavily loaded, especially if the ship's master should double up on his lines in event of a storm. The use of limit load bollards for these dolphins might prove wise. The author suggests that in the present case this load would be 100 tons per bollard. The long leads required to run the mooring lines for some of the ships might indicate a desirability for power capstans on the mooring dolphins. In the case of tankers of 130,000 DWT or more, the overhang of the bow of the vessel also becomes rather large when isolated breasting structures are used.

In addition to the preceding points regarding the flexibility of such a continuous face wharf to handle a large range of ships, there is a definite advantage to such a wharf when the number of berths to be made available is greater. If the required time in port can be reduced by improving the ease of berthing, deberthing or mooring, it may be possible to not only reduce the transport cost, but also to reduce the number of berths required for a given throughput. In this connection, most ship owners have indicated a desire that a ship be turned around in less than 24 hr regardless of size.

Berthing Forces.—The author makes a strong argument in favor of his opinion that large tankers do not necessarily produce impact forces exeeding those of smaller size ships. His reasoning follows closely that of P. Leimdorfer, F. ASCE, as presented at the 1957 congress of the P.I.A.N.C. in London. This opinion is also held by a number of other persons, especially those who have tried to observe actual ship velocities. Leimdorfer noted in his report, however that the Stockholm harbor is most sheltered, being located 27 miles from the sea with hardly any currents, maximum wave heights of $2\frac{1}{2}$ ft, and moderate winds. He also stated that all vessels are accompanied by tug boats within the harbor waters, and he warned that conditions existing in the Stockholm harbor can hardly be generalized for use in other ports.

The Bureau of Yards and Docks has made field measurements of mooring forces of full sized ships in correlation with the effects of winds and currents. These tests were being made with various vessels but especially with aircraft carriers at the Navy Yard piers in Bremerton, Washington and Terminal Island, California. The results of the tests indicated low line pull and in some cases, no pressures were registered against the wharf structure. Due to the surprisingly low values measured, the Bureau of Yards and Docks considered these values to be unrealistic. They had also intended to measure impact velocities of ships berthing at wharfs but these tests have not yet been made to the writer's best knowledge.

One method of determining the ship's approach velocity is to assume that this is due solely to wind on the vessel acting over a certain period of time and being in turn resisted by the drag of the water on the moving ship. P. Callet in his report to the 1953 conference of the P.I.A.N.C. has presented this analysis quite well. From his analysis, several reasons are apparent why a large vessel might not have as high an approach velocity as a smaller ship. First, the force of a gust on a large surface is not as great as on a smaller one. Second, the drag of the water which must pass under the bottom of the ship drifting broadside towards the pier varies inversely as the square of the clearance of the ship above the bottom and directly with the length of the vessel. In addition, a ship approaching a pier directly on its beam would have a much greater drag in this respect than one approaching at a considerable angle to the pier. This could explain some of Leimdorfer's observations. Trapping of water between a solid face wharf and the ship could also produce results similar to those observed by the Bureau of Yards and Docks. Third, the maximum wind velocity to be considered is the value beyond which the master deems it wise to put off the maneuver. This velocity obviously depends on the ship and on the characteristic of the harbor, especially on the position of the mooring structure with respect to the wind. The master's knowledge of the fender system's effectiveness may also have a bearing on his handling of the ship.

A few large vessels are constructed these days without model tests concerning the propulsion and power requirements, it would seem to be possible to make some of these models self-propelled and to conduct tests of the approach velocities and impact forces for simulated berthing operations. One objection which might be raised in this connection is that since the time scale ratio must be proportional to the square root of the length scale ratio, in order to conform to Froudes law, a ship model must be made to react rather quickly. This has proven to be no real handicap, however, and the Wageningen Scheepsbouw Proefstation in The Netherlands for example is performing such sea handling tests in their Sea Keeping Laboratory. The test conditions simulate a ship in the open sea and may include both waves and wind from any direction, but as far as the writer knows, no tests are to be made simulating shallow water or the berthing maneuvers in which the structural engineer as a wharf designer is interested. Other laboratories have run navigational tests on self-propelled model ships in harbor or river models, but here again, so far as is known, no attempt has been made to determine berthing forces.

This subject of the ship's velocity at impact is an item of study at the twentieth International Navigation Congress to be convened in Baltimore on September 11, 1961. It is expected that additional light will be shed on this subject at that time.

Fender Systems.—The author emphasizes that the rubber sandwich buffer is perhaps the most efficient type of spring for absorbing energy. A pretensioned steel spring, however, could be used to yield a lower reaction than a comparable rubber sandwich buffer for a given energy absorption and deflection. Steel springs on the other hand are subject to breakage of the structural guides since whenever energy must be absorbed by a rigid object, the reaction approaches a large value.

A fender system which is satisfactory for a 100,000 DWT tanker tends to be too stiff for a 2,000 DWT coastal tanker. This might call for a two element system such as a combination of rubber rolls and rubber sandwich buffers, or pehraps rubber sandwich buffers with a design deflection of 24 in. to 30 in. might be used. The English licensee of these fenders has completed tests on models representing fenders with as much as 30 in. of deflection without observing any tendency toward instability. They have also made overload tests indicating that these fenders can be subjected to at least a limited number of overload deflections without signs of distress. It would seem wise to incorporate the advantages of both high deflection and provision for overload deflection in future designs even though at some additional cost.

JOSEPH H. FINGER.[12]—Having carefully followed through the author's computations on the fender system designed for a 105,000 DWT tanker and the resultant reactions and deflections for each buffer shown in the load diagram (Fig. 13), it was difficult to understand how he could reach the conclusion that approximately 30 ft of the bow of the vessel will be in actual contact with the fender system at the time of maximum deflection.

Referring to Fig. 13, it is noted that, with the acting forces (13 kips per lin ft) located at about the center of the breasting platform, the end buffers deflect about 1/2 in. with a reaction of 2.78 kips and the adjacent buffers deflect about 7 3/8 in. with a reaction of 38.82 kips, successive buffers having respectively greater deflections and reactions toward the center.

Consider the steel wale which will be capable of transmitting these loads to the buffers. The section modulus of such a steel member, computed on the basis of an elastic limit of 33 kips per sq in., should not be less than 170 sq in. A steel member having a section modulus of this magnitude would be close to a 18 WF 96, with a moment of inertia equal to 1674. Considering such a steel member to be a continuous wale in the fender structure, and recognizing the stiffness of such a steel member, it is most difficult to understand how the beam could ever deflect 18 in. from the first to the fourth buffer without exceeding its elastic limit and resulting in a permanent deformation. It must, therefore, follow that if a number of buffers are connected by a continuous steel wale, the stiffness of the wale becomes the limiting factor of the resiliency of the buffers, and consequently of the fender system as a whole.

Another approach to the case shown in Fig. 13, assuming that a 18 WF 96 steel member were used as a continuous wale, is to accept the fact that the two center buffers were deflected approximately 18 in. Adjacent buffers, considered away from the center, would be deflected 17 in., and 16 in., and 15 in., respectively, if the elastic limit of the steel wale were not to be exceeded. To obtain such deflections on the buffers would require an acting force far greater than that shown (13 kips per lin ft), with a contact surface greater in length

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than the 30 ft determined by the author. Under such conditions the fender cannot assume the shape of the bow of the super tanker.

In fact, due to the stiffness of the wale, the contact of the vessel with the fender, under the berthing conditions given for Case II, will be restricted to much less than the 30 ft shown in Fig. 13. Consequently the magnitude of the acting force, per lineal foot, will be much higher than the 13 kips used by the author for his load diagram. The result of this situation will be that the buffers, applied as shown, will never reach the deflections obtained when tested singly in the laboratory.

Despite the discrepancies previously cited in the design of the fender system selected as the most suitable for the berthing conditions described, the author is to be congratulated on the thoroughness of his analysis of site conditions. His determination of the kinetic energy which a super tanker may impart to a mooring platform, considering wind direction and velocity, wave action, tides and currents, as well as velocity and angle of approach, were clearly presented. Having carefully compiled the basic data on which an adequate fender system must be based, the author then briefly analyzed the gravity, spring, and certain types of rubber fender systems, discarding them all in favor of the rubber-sandwich (Raykin) system. Although in actual operation the system selected may be affording satisfactory service, its adequacy to fullfill the design criteria shown, is questioned.

To go one step further, since the author states in his conclusion that, "it is hoped this paper will lead to discussion . . . . . so necessary for the proper design of modern marine terminals," attention is directed to another type of resilient fender system which the author has apparently overlooked. This is the Retractable Fender System, in use for the past 7 yr on the most diversified types of pier structures with excellent results, but without the massiveness or complicated suspension system which the author considers objectionable in his analysis of gravity and inertia fenders. The retractable system is based on absorbing kinetic energy by utilizing the gravity of the frame, the frictional resistance obtained from the travel of the fender on an inclined plane, and the friction between the upward moving fender and the hull of the vessel. Depending on site conditions and the amount of kinetic energy to be absorbed, the retraction (or length of travel) may vary from 8 in. to 48 in. Various aspects of this system have been more thoroughly described elsewhere.[13,14,15]

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13 "New Retractable Marine Fenders System," by Palmer W. Roberts and Virgil Blancato, Proceedings, ASCE, Vol. 84, No. WW1, January, 1958.

14 "A Bresting Dolphin for Berthing Supertankers," by John M. Weis and Virgil Blancato, Proceedings, ASCE, Vol. 85, No. WW3, September, 1959.

15 Dock and Harbor Authority, December, 1960.

BEHAVIOR OF BEACH FILLS IN NEW ENGLAND[a]

Discussion by A. C. Rayner

A. C. RAYNER.[11]—The author is to be commended for the work he has accomplished in compiling useful data regarding the use of beach fills as a shore protection measure. As he has stated, the periods of study since initial placement of the beach fill are generally short. During the initial period losses are frequently difficult to distinguish from changes due to profile adjustments. Longer periods of observation should furnish more realistic rates of losses in many cases. Additional data to June, 1960 for Prospect Beach made available to the Beach Erosion Board since writing of the original paper indicate an average annual loss of approximately 13,000 cu yd from between the planes of mean high and mean low water. This is an indication of the probable annual nourishment requirement, rather than the indication of no nourishment requirement based on data to June, 1959. It is hoped that surveys will be continued to provide data on behavior of beach fills over longer periods.

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[a] February, 1961, by Harry S. Perdikis (Proc. Paper 2744).
[11] Chf., Proj. Development Div., Beach Erosion Bd., Washington, D. C.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number [illegible]. [illegible] sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols [illegible] Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics [illegible], Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary [illegible] Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping [illegible], and [illegible] (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

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c. Discussion of several papers, grouped by divisions.